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DISCUSSION, PROCEEDINGS V. 55

Discussion of papers published in the April through June, 1959, JOURNALS appears in this issue, along with title page, errata, contents, and indexes for Proceedings V. 55, thereby completing this volume. Discussion of papers published in the July through September, 1958, JOURNALS appeared in March, 1959; discussion of papers published October through December, 1958, appeared in June 1959; discussion of papers published January through March, 1959, appeared in the Part 2, September, 1959, issue. Discussion pages in this issue are numbered continuously from the final page of the Part 2, September issue.

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ERRATA and ADDENDA

The following additions and corrections should be made in the indicated papers and reports of *Proceedings* V. 55, July 1958 through June 1959.

Title No. 55-2 "Recommended Practice for Design of Concrete Pavements (ACI 325-58)"

p. 39—Eq. (13) should read:

$$L_T = \frac{2f_s A_s'}{u\ Q} + 3 \text{ in.}$$

* * * *

Title No. 55-10 "Pressures on Formwork"

p. 189—The equations in the middle of the page should read:

$$p = C_1 \left(1 + \frac{C_2 R}{T}\right)$$

and

$$p = 150 \left(1 + \frac{60R}{T}\right)$$

* * * *

Title No. 55-16 "Origin, Evolution, and Effects of Air Void System in Concrete. Part 2—Influence of Type and Amount of Air-Entraining Agent"

p. 271—First line of type above Fig. 6 should read in part, "Fig. 6 shows the correlation between $\bar{L}$ and . . ."

* * * *

Title No. 55-29 "Folded Slab Construction" (see p. iv)

* * * *

Title No. 55-59 "Commentary on Concrete"

p. 928—Reference is made to the Dome of the Public Library in Melbourne and attributes its design to John Monash. Our attention has been called to the fact that while a design was made by Sir Monash, it was not used. The structural design should be attributed to Trussed Concrete Steel Co., London.

* * * *

Title No. 55-62 "Role of Cement in the Creep of Mortar"

p. 970—In Table 3, column headed "120 days, C_{120}" under "Stress of 2150 psi" for Cement No. 11, change 58 to 48.

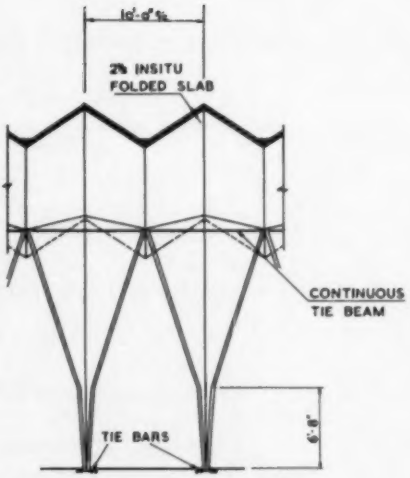
p. 975—In Table 7, co'umn headed "28-day cylinder strength, psi" for Cements 2, 2A, and 2B, change 5350, 5400, and 5120 to 3770, 4000, and 3900, respectively.

p. 977—The last sentence of the fifth paragraph from the top of the page should be corrected, in part, to read "... usual range of working stresses.[25]"

* * * *

Title No. 55-29 "Folded Slab Construction"

The accompanying illustrations represent an addition to the paper. These drawings show two cross sections of the assembly hall described at the bottom of p. 459. The folded slab not only forms the roof in this construction, but it is carried around the side elevation and in fact replaces a rigid frame. Because of headroom it is carried down only to a height of 6 ft 8 in., with the vertical portion of the frame carried through to the bottom, and made rigid enough to take the bending moments caused by the horizontal thrust.



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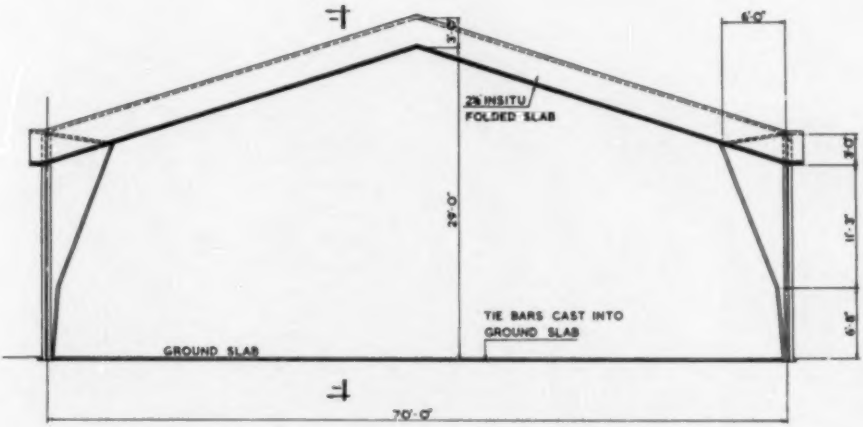

* * * *

Disc. No. 55-62 "Role of Cement in the Creep of Mortar"

p. 1558—The first sentence of the second paragraph should read, in part, "Tensile strength, σ_r, of concrete rings subjected to restrained shrinkage, δ, was determined; . . ."

p. 1559—In the next to last line ε_s should read $\bar{\varepsilon}_s$.

* * * *

Title No. 55-71 "Cylindrical Shell Analysis Simplified by Beam Method"

p. 1185—add the following to the Notation:

p = load per sq ft of roof surface for dead load
= load per sq ft of horizontal projection for uniform load
w = load per longitudinal foot of shell
= $2p_{DL}\, r\phi_E$ for dead load
= $2p_{LL}\, r\sin\phi_E$ for uniform load

p. 1189—in Eq. (15), 3.030 should be 3,030.

p. 1190—in Eq. (16), 3.030 should be 3,030.

p. 1191—in Fig. 9, the scale for T_ϕ should be 6, 4, 2, 0, 2, 4 instead of 3, 2, 1, 0, 1, 2.

* * * *

Title No. 55-79 "Effect of Powdered Minerals and Fine Aggregate on the Drying Shrinkage of Portland Cement Paste"

p. 1306—For the reference in the last line read 15 instead of 13.

p. 1315—The formula under the figure should be corrected to read

$$\log\triangle S'_{o\text{-}E} = \log\triangle x_{o\text{-}E} + a'R + b'$$

* * * *

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Discussion of a paper by José J. Rodriguez, Albert C. Bianchini, Ivan M. Viest, and Clyde E. Kesler:

Shear Strength of Two-Span Continuous Reinforced Concrete Beams*

By G. BROCK and R. TAYLOR

By G. BROCK†

This discussion will be restricted to an examination of the authors' test results and the presentation of some complementary data. Although the writer would describe "shear failures" in a different way, it seems preferable to reserve this description for a future paper.

Unrealistic design methods for reinforced concrete beams persist chiefly because shear and bending are treated as independent phenomena to be designed for separately. Progress in ultimate load design can be made if we regard shear as a factor which modifies the flexural resistance.

In recent years the ultimate load approach to frame design has been facilitated by the methods developed for steelwork by J. F. Baker and his associates.[16] They have shown that the collapse load of a frame can be determined on the assumption that sufficient plastic hinges will develop to convert the structure to a simple mechanism. But the reinforced concrete engineer should follow this approach with caution. He must be sure that his typical structures can undergo the deformations necessary for this kind of collapse.

The effects of shear seem to be more important in reinforced concrete than in structural steelwork. They may cause collapse at loads much smaller than those necessary for the development of a plastic hinge mechanism. It will be interesting, therefore, to compare the collapse loads observed in experiments with those calculated by plastic theory. Where shear is important we should expect to find that the plastic theory seriously overestimates the strength.

AUTHORS' RESULTS EXAMINED BY PLASTIC THEORY

To calculate the collapse load of a continuous beam we must know the maximum positive and negative moments it can resist. For under-reinforced rectangular beams these moments can be calculated from Whitney's formula,[17]

---

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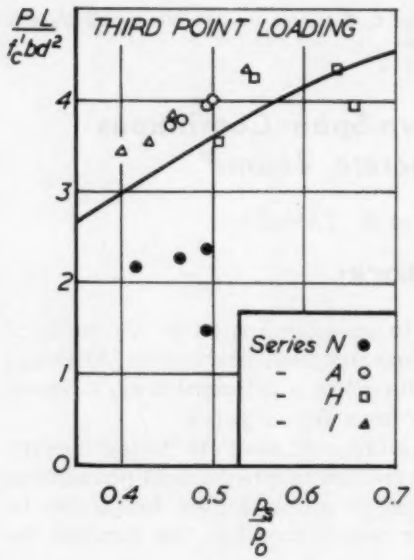

Fig. A—Variation of collapse load with tensile reinforcement index at midspan. Comparison of authors' results with plastic theory.

$$M/f_c' bd^2 = 0.456\, p/p_o\, (1 - 0.268\, p/p_o)$$

where $p_o = 0.456\, f_c'/f_y$.

Table 2 shows that all the beams reported were under-reinforced. The proportion of tensile reinforcement over the central support was always 67 percent greater than that provided in the span (p_s). This means that both the ultimate hogging and sagging moments, and hence the collapse load, can be related to the reinforcement index at midspan (p_s/p_o). On this basis ultimate load calculations have been made for the three loading patterns investigated, and the results are shown by continuous lines on Fig. A and B. The test results have been plotted as single points in the same dimensionless form.

Comparison shows little discrepancy between experiment and plastic theory, except for the beams of Series N, which had no web reinforcement. All Series N tests fell short of the calculated plastic strength. The deficiency was greatest with midpoint loading, which by conventional calculations would appear to be governed least by shear.

On the other hand there appears no significant difference between the results of Series A, H, and I where different amounts of web reinforcement were employed. It seems that a modicum of web reinforcement is necessary to control shear but that further amounts do not improve the strength.

The importance of the longitudinal reinforcement index on the collapse load is revealed both by theory and experiment in conclusive agreement.

EFFECT OF SPAN-DEPTH RATIO

All the beams tested by the authors had a constant L/d value of 8.15. T. J. Fowler, K. H. Hohenkerk, and G. R. Leake, final year undergraduates at the University of Birmingham, working under the writer's direction and using his model technique,[18] tested 30 two-span rectangular beams with varying L/d.

Each beam had the same number of straight reinforcing rods at the top and bottom of the sections. Each rod ran the full length of the beam, which overhung the outer supports sufficiently to provide full anchorage. No web reinforcement was used, and the longitudinal reinforcement index was arranged to be either 0.2, 0.7, or 1.20 in each beam.

To simulate well distributed loads, such as are often encountered in practice, the test loads were applied uniformly at 16 equally spaced points. The experimental results were shown on Fig. C, in terms of L/d, for the three values of p/p_o investigated. The theoretical plastic collapse loads are also plotted. They do not involve L/d as a variable.

It will be noted that in all tests with $p/p_o = 0.2$ the ultimate load conformed with plastic theory. In those with $p/p_o = 0.7$ it fell away from the plastic value as L/d was reduced below about 15. The over-reinforced beams ($p/p_o = 1.20$) did not quite reach the plastic theory collapse load even at L/d values of about 22.

These tests illustrate clearly that the need for shear reinforcement increases as the L/d ratio is decreased, especially in heavily reinforced beams.

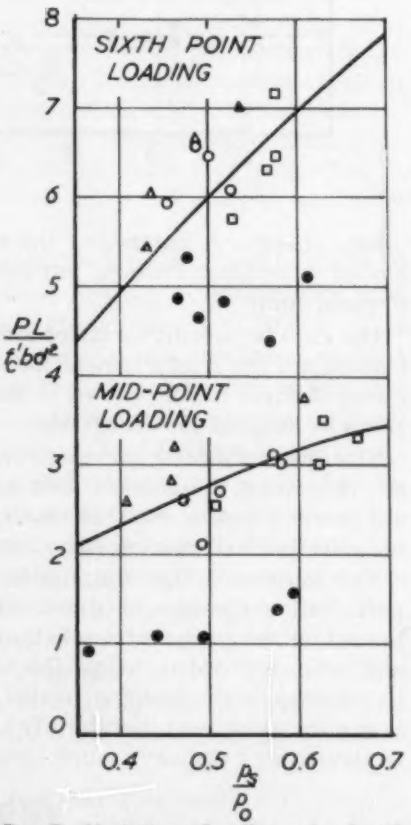

Fig. 8—Variation of collapse load with tensile reinforcement index at midspan. Comparison of authors' results with plastic theory.

CONCLUSIONS

The importance of shear on any test beam can be gaged by the discrepancy between the actual collapse load and that calculated from

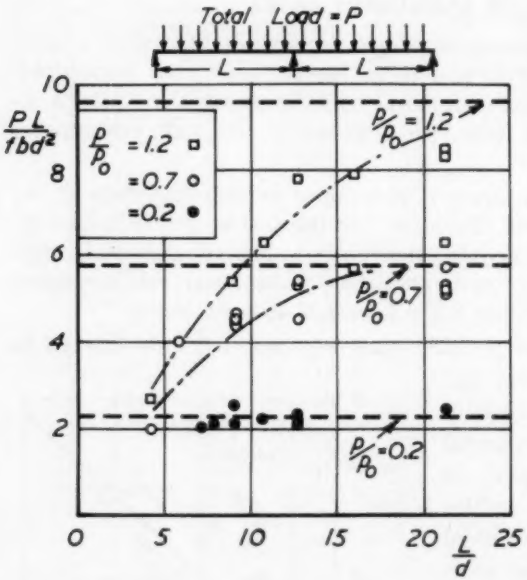

Fig. C—Effect of L/d on collapse load of model reinforced beams under 16 point loads. Horizontal broken lines show collapse loads indicated by plastic theory

plastic theory. A number of the experiments reported by the authors looked to be shear failures, but calculations show they had reached the flexural limit.

The most important variables affecting failure for a given geometry of loading are the reinforcement index p/p_o and the span-depth ratio. The effect of these can be shown in dimensionless charts (Fig. A-C) which could be adapted for design use.

The effect of adding some web reinforcement is shown to be important but increasing the amount does not always affect the collapse load to any marked degree. Further research to determine the minimum amount for effective web reinforcement would appear worth while.

The common design assumption that the strength of a beam is proportionate to the amount of web reinforcement appears to be unfounded. Indeed, it seems doubtful whether the usual practice of proportioning web reinforcement to follow the working load shear force diagram can be justified. If the designer related his web reinforcement to the p/p_o and L/d ratios and spread it uniformly along the beam, he might obtain sounder structures[19] and save much time in the design office and on the site.

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By R. TAYLOR*

The authors have presented such a wealth of data on the shear strength of reinforced concrete beams that it is difficult to make adequate comment on the whole paper. There are, however, several isolated points which the writer would like to make.

Considerable difficulty has been caused in the past because the various investigators have adopted different definitions for diagonal cracking. The authors appear to have added somewhat to the confusion since, with their definitions, the diagonal tension cracking load is not necessarily the load at which the first diagonal tension crack forms but is that at which the diagonal tension crack has extended sufficiently into the beam to cause an appreciable redistribution of internal stresses. Such terminology is confusing, apparently even to the authors, since they subsequently refer to diagonal tension cracking loads as being the loads at which the first diagonal tension crack formed.

The definition of the "diagonal tension cracking load" as it applies to beams without shear reinforcement does, however, coincide with that adopted in the many recent papers from the University of Illinois. This is important as it is now largely recognized that it is this stage, and not collapse, that must mark the failure criterion of such beams for practical purposes. There is, of course, often no difference between the diagonal tension cracking load as so defined and the collapse load for beams loaded so that the a/d ratio is comparatively large. That there is often a considerable difference between the two stages for beams loaded so that the a/d ratio is small has now been frequently demonstrated. What does not yet appear to be completely appreciated is that this difference is dependent on the load being applied to the beam at the top surface. The effect of the manner of loading on the shear strength of beams without shear reinforcement was first pointed out by Ferguson[20] in 1956. Ferguson suggested that when a beam is loaded in such a way that propagation of a diagonal crack is not inhibited by compressive stresses set up by a load block, e.g., by loading through transverse secondary beams or nibs framing into the beam, then the collapse load may be considerably reduced. He demonstrated this effect in a series of tests on beams with short shear spans. More recently tests have been carried out at the Building Re-

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Fig. D—Typical beam with loading and supporting nibs

search Station, England, to investigate this effect over a greater range of a/d ratios. The type of beam used in these tests is illustrated in Fig. D. The results of the tests indicated that:

(1) The diagonal cracking load is the maximum load that can be sustained by a beam when it is loaded and supported through transverse beams, irrespective of the length of the shear span.

(2) The manner of loading a beam has little influence on the diagonal cracking load, except perhaps for beams with short shear spans.

(3) The diagonal cracking load is only slightly influenced by the shear span, decreasing slightly with increase in shear span.

Since the loading of a beam through secondary beams often occurs in practice these results are further evidence, if any were needed, that the diagonal cracking load must represent the failure criterion for beams without shear reinforcement. But the question now arises as to whether the manner of loading has any effect on the shear strength of beams with shear reinforcement. It would seem that it is necessary to answer this question before existing experimental data can be translated into design recommendations.

Continuity in reinforced concrete raises several interesting problems with regard to shear strength. One problem which the writer has been concerned with recently—again for beams without shear reinforcement—is the effect of the order of loading the spans of a continuous beam. A study of the mechanism of diagonal cracking had suggested a possible influence. Except for beams with short shear spans, diagonal cracks generally develop from curving flexural cracks. The extent of a flexural crack above the soffit of a beam will generally increase with increasing bending moment, although in the later stages of development the change in extent is only small. If the major part of the shear force is carried across a cracked section of a beam by the uncracked part of the concrete above the crack, it would seem that the higher the bending moment on a section the lower might be its shear-carrying capacity. However it is possible that the higher stresses in the compression zone compensate this effect to some extent by offering greater restraint to the propagation of the cracks due to shear. If this is so, in a beam in which flexural cracks have been formed (by the application of a high bending moment) there will be a loss of shear resistance if the stresses in the compression zone are subsequently reduced without alteration of the shear force. Such an

event might occur in a continuous beam by the separate loading of the spans.

To investigate this problem a short series of tests was made on five beams cantilevering past the two supports. The general arrangement for testing can be seen in Fig. E. The beams, 7½ in. wide by 10¼ in. deep, were supported at two positions 6 ft apart and load was applied by hydraulic jacks at three positions, that is at midspan and at points 2 ft outside each support. The jacks were operated individually, the load being applied through load cells by means of which the load was measured.

Three beams, identical in respect to reinforcement and concrete mix, were tested with this arrangement by loading initially at the central section and subsequently on each cantilever. With this type of loading, flexural cracks were initially formed at the soffit in the usual way, and the effect of the superimposed load system on these cracks was observed. In two of the tests the load applied at the center of the beam was approximately 90 percent of the average diagonal cracking load found for similar beams loaded centrally on a 6-ft span; in the third test the central load was 95 percent of this load.

The essential results for this discussion are:

(1) During the loading of the cantilevers, propagation of one or more of the original cracks occurred in all three tests in a way similar to that occurring in a simply supported beam under increasing load.

(2) In one beam, that loaded at approximately 95 percent of the diagonal cracking load of similar simply supported beams, diagonal cracking occurred by the development of one of the original flexural cracks—see Fig. E. (In the other two beams diagonal cracking also occurred in the internal shear spans, but developed from flexural cracks initiated at the top surface. Such cracking in a continuous beam is dependent only on the relative values of the positive and negative bending moments and resistance moments and is not pertinent to the discussion on the order of loading.)

In the tests on the other two beams, the three spans were loaded simultaneously, the magnitude of the loading being arranged so that diagonal cracking developed from flexural cracks formed at the soffit. In these tests the shear force at diagonal cracking was higher than that which caused diagonal cracking in similar beams loaded at midspan only. Thus in interpreting test data it is important to realize that this increase

Fig. E—General arrangement for testing beams cantilevering past the supports

is dependent on the simultaneous loading of the spans. When a span is loaded prior to the adjacent spans being loaded there may in fact be a decrease in the critical shear force, although this decrease is only small.

REFERENCE

20. Ferguson, Phil M., "Some Implications of Recent Diagonal Tension Tests," ACI JOURNAL, V. 28, No. 2, Aug. 1956 (*Proceedings* V. 53), pp. 157-172.

Discussion of a paper by C. E. Proudley:

Qualification Plan for Ready-Mixed Concrete Plants*

By MILES N. CLAIR, E. F. LEWIS, JOSEPH J. WADDELL, and AUTHOR

By MILES N. CLAIR†

The excellent work done by Mr. Proudley and his associates in the North Carolina State Highway Commission is well known. The paper describes one of the procedures used to aid in obtaining good concrete. It is a fine idea and the paper is particularly valuable in that it gives in detail how the qualification plan is being carried out. One item however, that is normally of interest does not seem to be fully covered in the text of the memorandum, namely, the capacity of the plant and equipment based on their records of performance. Capacity of equipment is not always the same as actual production and a major factor for approval for some projects may be ability to deliver concrete of volume as well as quality.

It is noted that in the check list nothing is said about source, quantity, or quality of water. This is often an important factor where water is scarce. In areas colder than North Carolina information on facilities for heating aggregates and water is desirable.

The success of this plan would seem to depend not only how serious the state representatives are in applying the rules, but also the cooperation shown by the ready-mixed concrete producers, and the real complications might arise if the right of approval or basis of approval is challenged. A number of building codes have introduced required approval of ready-mixed concrete plants and the proposed revision of Part 26 on concrete of the Boston Building Code includes such a requirement. In these cases the codes are enforceable as laws and the ready-mixed producers must conform.

By E. F. LEWIS‡

Our ready-mixed concrete plants have operated under the North Carolina State Highway Commission plan for the past several years. Mr. Proudley did not say in his article that ready-mixed concrete plants which

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‡Member American Concrete Institute, President, F. D. Lewis & Son, Inc., Greensboro, N. C.

obtain the A-1 rating and agree to assign a "concrete technician" on every state job are allowed to place state highway concrete at any time and without an inspector.

We have operated on this basis for about 2 years and consider it an honor which we guard jealously. To meet this program we now have four concrete technicians licensed by the state highway commission. It has meant that our personnel have done considerable studying, and to pass the examination, know and practice the fundamentals of good concrete construction. This has carried thoughout our organization and has had a tremendous effect for the good on all concrete, whether state or private. There is a general tightening of concrete specifications by private engineers and architects, and the program of the state highway commission has brought about training in our personnel, enabling them to better meet all standards.

We have been somewhat disappointed in the participation by ready-mixed concrete plants over the state. It affords a plan to increase the ability of any plant to produce better concrete through the installation of proper equipment and training of personnel. Once you have brought your personnel up to the standards required for a concrete technician, they are eager on their own accord to learn more and to produce better concrete. We hope that more ready-mixed concrete producers will participate in the future. The production of better concrete is one of the best sales means that a producer can obtain.

By JOSEPH J. WADDELL*

The paper by Mr. Proudley presents an approach that should be more widely adopted. Wider usage would tend to improve ready-mixed concrete operations throughout the industry, forcing substandard plants to improve their facilities and operations.

One important aspect mentioned by the author is the subject of personnel training. Adequate training of personnel, both inspection and operating, is a problem that is constantly facing construction engineers. Especially when transit mixed concrete is permitted, properly trained operators are essential if anything approaching control is to be obtained.

It would be of considerable help to some readers if the author could expand his discussion of personnel training. Perhaps a list of the references used could be furnished, together with some discussion of the examinations that were given.

*Member American Concrete Institute, Knoerle, Graef, Bender & Associates, Inc., Chicago, Ill.

AUTHOR'S CLOSURE

The training program for "Registered Concrete Technicians" mentioned in connection with the qualification plan has created more comment than any other item in the paper. Expansion of the training school to include some time for those who are now registered will be a new feature during the coming winter program. This will take up behavior of concrete under various conditions and the analysis of faults or failures which are most likely to occur where control has not been adequate.

Since writing the article, many more technicians have been registered. As pointed out by Mr. Lewis, much serious study is required to qualify and there are personnel of both the highway commission and the industry constantly preparing for examination. It is a slow process since it requires considerable preliminary reading and much home work with the computations for proportioning and adjustment of concrete mixtures.

To answer Mr. Waddell's suggestion in part, all of the texts used in the instruction are available from the Portland Cement Association, the National Ready Mixed Concrete Association, and the aggregates associations. For those who may wish to secure them, they are:

1. "Estimating Proportions for Concrete," *NSGA Circular* No. 68 and *NRMCA Publication* No. 69, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D.C.
2. "Control of Quality of Ready-Mixed Concrete," *Publication* No. 44, National Ready Mixed Concrete Association, Washington, D.C.
3. "Specifications and Test Methods for Ready-Mixed Concrete," *Publication* No. 47, National Ready Mixed Concrete Association, Washington, D.C.
4. "Design and Control of Concrete Mixtures," Portland Cement Association, Chicago.
5. "A Method of Proportioning Concrete for Strength, Workability and Durability," *Bulletin* No. 11, National Crushed Stone Association, Washington, D.C.
6. "Recommended Practice for Selecting Proportions for Concrete," *Publication* No. 45, National Ready Mixed Concrete Association, Washington, D.C. (A reprint of ACI Standard ACI 613-54.)
7. "Sampling and Testing Ready-Mixed Concrete," *Publication* No. 66, National Ready Mixed Concrete Association, Washington, D.C.

The questions in the examination are quite comprehensive and have been criticized for being "too tough." We require a 100 percent grade on the examination before certifying but this is not as bad as it sounds. We are anxious to create registered technicians and if the applicant will continue with additional examination, mostly verbal, after his first trial, our concrete engineers who conduct the examinations will assure us that the prospective technician fully understands every question and answer. We have supplied interested persons and organizations with sample copies of the written portion of the examination on many occasions.

Although it has been gratifying to have the approval plan for ready-mixed concrete plans received so enthusiastically there is one serious

deficiency in it. The quality of the end product is not rated! Although it is unlikely that a fully qualified technician with a well equipped plant will turn out inferior concrete, there is no assurance from the list of approved plants that they will be consistently reliable in the uniformity of delivered concrete.

During the past construction season the compression test strengths representing concrete delivered to state highway projects have been analyzed for coefficient of variation and average strength. The average strengths from the 50 plants which furnished state jobs during this period (8 months) ranged from 3360 to 5810 psi for 28-day strengths (6 bags per cu yd cement required). Minimum strength of 3000 psi is the specification. The coefficients of variation range from a low of 3.6 percent to a high of 25.6 percent. Incentive is a powerful influence toward improvement and it is our intention to work out a plan whereby those who have excellent uniformity and ample strength will be permitted a more favorable cement factor than the plants which have a high coefficient of variation even though the average strength exceeds the required strength by a wide margin. The items requiring consideration in such a plan are many and controversial but it is our belief that such incentive will result in over-all improvement quicker than an educational program alone.

Discussion of a paper by James Chinn:

Cylindrical Shell Analysis Simplified by Beam Method*

By MILO S. KETCHUM, A. L. PARME and H. W. CONNER, A. SIEV, ANTON TEDESKO, ALFRED ZWEIG, and AUTHOR

By MILO S. KETCHUM†

The author has demonstrated that the beam method for barrel shells is quite accurate for the design of interior barrels of multiple barrel structures. The development of formulas is useful only for circular barrels of constant thickness. However, the advantages of the beam method extend far beyond the use for circular barrels of constant thickness.

The writer would like to list and discuss a few of these advantages:

(1) The beam method can be applied to a barrel of any cross section and variable thickness; thus barrel shells are not limited to circular shapes of constant thickness only.

(2) It can take into account a concentrated line load in determining the transverse bending moments. This has been demonstrated in a recent book by Chronowicz.‡

(3) The barrel shells may be ribbed in either direction.

(4) The beam method is not necessarily limited to a straight line distribution of longitudinal stress. Any distribution can be used. The procedure is to select two basic patterns of stress and then solve for the combinations of patterns which satisfy statics. This device is particularly useful as a method of correcting the theory of elasticity solution for circular barrels with nonuniform cross section. The pattern of longitudinal stress is taken from the elasticity solution for uniform cross section and is adapted to the nonuniform section.

The writer has used the beam method to study the stresses in hyperbolic paraboloidal structures. For sections cut through an element of the shell, the distribution of longitudinal stress is initially assumed as a pattern with constant stresses of one sign in the shell and the other sign concentrated at the stiffening rib. The basic simplified equation for the hyperbolic paraboloid can be reproduced by this device.

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†Member American Concrete Institute, Ketchum, Konkel & Hastings, Denver, Colo.

‡Chronowicz, Albin, *The Design of Shells*, Frederick Ungar Publishing Co., New York.

Several writers have studied the effect of changes in the stress pattern on the design of barrel shells. For example, Kirchner* has demonstrated that there is only a small variation in transverse moments for a number of different distributions of longitudinal moment.

Jenkins† has presented a method of determining the stresses in barrel shells based on minimum strain energy. Several possible longitudinal stress patterns are selected and the ratios of these patterns are determined from minimum strain energy.

The beam method for barrel shells has many uses and should be studied by every designer of shell construction because it provides an insight on the structural action of shells that cannot be obtained from the more accurate but more tedious theory of elasticity solutions.

By A. L. PARME and H. W. CONNER‡

Dr. Chinn is to be complimented on the neat and concise arrangement of the formulas obtained on the basis of conventional methods of analysis dealing with the internal forces and moments occurring in a member of curved cross section. It has been recognized by many that within certain proportions the stresses obtained by treating the shell as a beam agree with those obtained by a rigorous solution of the eighth order differential equations derived from the elastic behavior of space structures.§ Full exploitation of this fact has been difficult to realize principally because limits within which the approximate beam solution is valid could not be clearly established. It is readily apparent that when the chord width exceeds greatly the span, the analysis of a shell by the beam theory is meaningless. For this condition the longitudinal carrying action is confined to a zone near the valley, and the upper portion of the shell is relatively free of longitudinal forces. It is not so readily evident that some degree of inaccuracy may hold as well for long narrow shells.

The author has correctly stated the assumptions upon which the beam analogy is predicated. Unfortunately, the assumptions give no clue as to the range of applicability. Under all conditions, the presence of transverse moments indicates that at a given cross section, the edge and crown will deflect a different amount. Thus the first assumption can never be exactly fulfilled. But complete satisfaction of this assumption is not needed, since it is the relative deflection of the valley to the crown that accounts for significant departure of stresses in a shell from that indicated by the beam theory. As yet there is insufficient data avail-

*Kirchner, G., "Zur Berechnung langer Zylinderschalen," *Der Bauingenieur* (Berlin), V. 34, No. 9, Sept. 1959, pp. 331, 336.

†Jenkins, R. S., "A Variational Method for Design of Cylindrical Shells," *Second Symposium on Concrete Shell Roof Construction*, Teknisk Ukeblad, Oslo, pp. 157-167.

‡Members American Concrete Institute, Manager and Structural Engineer, respectively, Structural and Railways Bureau, Portland Cement Association, Chicago, Ill.

§*A Structural Examination of Barrel Shell Behavior*, Portland Cement Association, Chicago.

able to determine at which ratio a departure from the linear distribution of stresses occur.

To check as to whether the second assumption is complied with is almost impossible without recourse to shell analysis. The deflection at midspan could be used as a rough guide to determine how much longitudinal bending occurs. There is, however, a shortcoming to relying too greatly on this approach. Generally the maximum longtudinal bending and twisting moments occur near the corners. In this area they do not bear a direct relationship to the midspan deflection. Thus values obtained from midspan deflection might be misleading. Fortunately, the magnitude of the longitudinal bending is seldom critical.

In spite of these deficiencies the beam method is a useful and handy procedure for analyzing shells if due caution is taken of its limitations. It has one tremendous merit in that the behavior of a shell can be expressed in terms of the product of constants which depend only on ϕ_k and various multipliers. The advantage of this is that a compact table with a single variable can be prepared to cover a large range of cases.

Although the calculations entailed by equations presented by Dr. Chinn are considerably less complicated than those engendered by the more exact shell theory, they require nevertheless a high degree of accuracy. As an aid to the designer, Tables A and B are presented which have been computed on the basis of Eq. (1) to (21). The values listed are the internal forces at a cross section in multiple barrels for dead and uniform load and for unit symmetrical horizontal displacements of the edges. As a matter of information, the tables were computed by means of an electronic computer employing 12 significant figures. This accuracy was required since studies of the sensitive nature of the equations indicated that in some cases even with ten significant figures errors of 10 percent could accrue in the final answers. The coefficients listed are only given to four significant digits since slide rule accuracy is sufficient in their use.

The determination of the internal forces is readily evident from the formulas given in Table A for forces T_x, T_ϕ, S, and M_ϕ. The results given by the operations indicated inside the first bracket will coincide exactly with the numerical evaluation of Eq. (1) to (21). However, to extend the usefulness of the beam method a correction term is given in the second bracket in the expression for T_ϕ and M_ϕ.

For short shells, i.e., one whose ratio of span to chord width is small, and small central angles, the edges of a single shell tend to deflect inward. In contrast to this in long shells, when r/l is less than about 0.25 the reverse behavior takes place when a uniform load is applied. In this instance a flattening of the shells occurs causing the edges to deflect outward. This in some cases greatly alters the distribution of transverse moment. This flattening being due primarily to the direct

TABLE A — VERTICAL LOADS

$$T_x = \frac{l^2}{r}\,[p_u \text{ Col. } (1) + p_d \text{ Col. } (5)]$$

$$T\phi = r\,[p_u \text{ Col. } (2) + p_d \text{ Col. } (6)] + \frac{p_u t^2}{6r}\,[\text{Col. } (9) \times \triangle H/(p_u r^2/Et)]$$

$$S = -l\,[p_u \text{ Col. } (3) + p_d \text{ Col. } (7)]$$

$$M\phi = r^2\,[p_u \text{ Col. } (4) + p_d \text{ Col. } (8)] + \frac{p_u t^2}{6}\,[\text{Col. } (10) \times \triangle H/(p_u r^2/Et)]$$

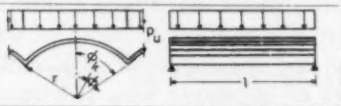
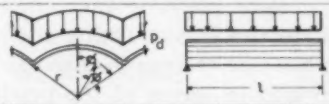
		Uniform transverse load				Dead weight load			
									
ϕ_k, deg	ϕ	T_x (1)	$T\phi$ (2)	S (3)	$M\phi$ (4)	T_x (5)	$T\phi$ (6)	S (7)	$M\phi$ (8)
	ϕ_k	−6.010	−1.411	0.000	−0.00292	− 6.167	−1.433	0.000	−0.00309
	$0.75\phi_k$	−4.875	−1.189	2.211	−0.00112	− 5.003	−1.205	2.269	−0.00118
22.5	$0.50\phi_k$	−1.482	−0.614	3.533	0.00232	− 1.521	−0.615	3.626	0.00245
	$0.25\phi_k$	4.137	0.049	3.084	0.00235	4.245	0.065	3.165	0.00249
	0	11.927	0.361	0.000	−0.00662	12.239	0.384	0.000	−0.00702
	ϕ_k	−4.855	−1.402	0.000	−0.00353	− 5.012	−1.430	0.000	−0.00378
	$0.75\phi_k$	−3.937	−1.182	1.985	−0.00135	− 4.064	−1.202	2.049	−0.00145
25.0	$0.50\phi_k$	−1.193	−0.612	3.170	0.00280	− 1.232	−0.613	3.273	0.00300
	$0.25\phi_k$	3.342	0.044	2.765	0.00282	3.451	0.064	2.855	0.00304
	0	9.617	0.347	0.000	−0.00797	9.929	0.374	0.000	−0.00857
	ϕ_k	−4.000	−1.393	0.000	−0.00417	−4.158	−1.426	0.000	−0.00453
	$0.75\phi_k$	−3.242	−1.175	1.799	−0.00159	−3.370	−1.199	1.869	−0.00173
27.5	$0.50\phi_k$	−0.980	−0.609	2.871	0.00331	−1.018	−0.610	2.985	0.00360
	$0.25\phi_k$	2.755	0.038	2.503	0.00332	2.863	0.063	2.602	0.00363
	0	7.908	0.331	0.000	−0.00938	8.220	0.363	0.000	−0.01025
	ϕ_k	−3.350	−1.383	0.000	−0.00482	−3.508	−1.422	0.000	−0.00533
	$0.75\phi_k$	−2.714	−1.166	1.643	−0.00183	−2.842	−1.195	1.720	−0.00203
30.0	$0.50\phi_k$	−0.817	−0.606	2.622	0.00384	−0.856	−0.607	2.746	0.00424
	$0.25\phi_k$	2.308	0.032	2.284	0.00383	2.417	0.061	2.392	0.00426
	0	6.608	0.314	0.000	−0.01082	6.920	0.352	0.000	−0.01204
	ϕ_k	−2.844	−1.372	0.000	−0.00548	−3.002	−1.418	0.000	−0.00618
	$0.75\phi_k$	−2.303	−1.158	1.511	−0.00207	−2.431	−1.191	1.595	−0.00235
32.5	$0.50\phi_k$	−0.691	−0.603	2.410	0.00438	−0.729	−0.603	2.544	0.00492
	$0.25\phi_k$	1.960	0.026	2.098	0.00434	2.069	0.060	2.215	0.00492
	0	5.596	0.297	0.000	−0.01227	5.908	0.339	0.000	−0.01393
	ϕ_k	−2.442	−1.361	0.000	−0.00615	−2.601	−1.414	0.000	−0.00707
	$0.75\phi_k$	−1.977	−1.148	1.397	−0.00232	−2.105	−1.186	1.488	−0.00268
35.0	$0.50\phi_k$	−0.591	−0.599	2.227	0.00491	−0.629	−0.600	2.372	0.00565
	$0.25\phi_k$	1.684	0.019	1.938	0.00484	1.793	0.058	2.064	0.00561
	0	4.794	0.278	0.000	−0.01370	5.105	0.326	0.000	−0.01591
	ϕ_k	−2.118	−1.349	0.000	−0.00679	−2.278	−1.409	0.000	−0.00800
	$0.75\phi_k$	−1.714	−1.138	1.298	−0.00255	−1.842	−1.181	1.396	−0.00302
37.5	$0.50\phi_k$	−0.510	−0.596	2.069	0.00544	−0.548	−0.596	2.224	0.00640
	$0.25\phi_k$	1.461	0.012	1.796	0.00532	1.571	0.057	1.933	0.00632
	0	4.146	0.260	0.000	−0.01509	4.458	0.312	0.000	−0.01796
	ϕ_k	−1.853	−1.335	0.000	−0.00742	−2.013	−1.404	0.000	−0.00897
	$0.75\phi_k$	−1.498	−1.127	1.211	−0.00277	−1.627	−1.176	1.315	−0.00337

TABLE A (Cont'd) — VERTICAL LOADS

		Uniform transverse load				Dead weight load			
ϕ_k, deg.	ϕ	T_x (1)	$T\phi$ (2)	S (3)	$M\phi$ (4)	T_x (5)	$T\phi$ (6)	S (7)	$M\phi$ (8)
40.0	$0.50\phi_k$	—0.443	—0.592	1.929	0.00595	—0.482	—0.592	2.095	0.00719
	$0.25\phi_k$	1.279	0.005	1.675	0.00578	1.389	0.055	1.819	0.00705
	0	3.616	0.241	0.000	—0.01641	3.928	0.297	0.000	—0.02066
	ϕ_k	—1.449	—1.307	0.000	—0.00853	—1.610	—1.393	0.000	—0.01096
	$0.75\phi_k$	—1.170	—1.104	1.065	—0.00316	—1.299	—1.165	1.183	—0.00408
45.0	$0.50\phi_k$	—0.343	—0.585	1.694	0.00688	—0.381	—0.583	1.882	0.00883
	$0.25\phi_k$	1.001	—0.011	1.468	0.00657	1.112	0.052	1.630	0.00854
	0	2.809	0.202	0.000	—0.01872	3.120	0.266	0.000	—0.02437
	ϕ_k	—1.160	—1.278	0.000	—0.00939	—1.322	—1.380	0.000	—0.01301
	$0.75\phi_k$	—0.935	—1.079	0.947	—0.00344	—1.065	—1.152	1.079	—0.00480
50.0	$0.50\phi_k$	—0.271	—0.578	1.504	0.00762	—0.308	—0.574	1.713	0.01054
	$0.25\phi_k$	0.802	—0.029	1.300	0.00714	0.914	0.049	1.481	0.01002
	0	2.232	0.164	0.000	—0.02042	2.543	0.234	0.000	—0.02871
	ϕ_k	—0.946	—1.242	0.000	—0.00989	—1.109	—1.367	0.000	—0.01506
	$0.75\phi_k$	—0.761	—1.053	0.849	—0.00358	—0.892	—1.139	0.995	—0.00549
55.0	$0.50\phi_k$	—0.217	—0.572	1.347	0.00807	—0.255	—0.563	1.578	0.01227
	$0.25\phi_k$	0.655	—0.048	1.161	0.00742	0.767	0.045	1.360	0.01144
	0	1.805	0.128	0.000	—0.02130	2.115	0.201	0.000	—0.03293
	ϕ_k	—0.783	—1.205	0.000	—0.00992	—0.947	—1.352	0.000	—0.01705
	$0.75\phi_k$	—0.629	—1.025	0.766	—0.00355	—0.761	—1.124	0.927	—0.00613
60.0	$0.50\phi_k$	—0.177	—0.566	1.213	0.00815	—0.214	—0.552	1.467	0.01398
	$0.25\phi_k$	0.543	—0.068	1.043	0.00734	0.656	0.042	1.261	0.01275
	0	1.481	0.095	0.000	—0.02118	1.790	0.167	0.000	—0.03688

internal forces (membrane stresses) and not due to bending, is not taken into account by the ordinary beam theory. To correct for this flattening Tables B and C are included. Table C gives the horizontal displacement due to a uniform load on the shell for various values of ϕ_k and r/l as computed from the membrane theory of shells. These values multiplied by the corresponding coefficients in Table B yield the correction which must be applied to M_ϕ and T_ϕ.

No correction is given for dead load, since this type of loading does not lead to any appreciable error.

The importance of the need for including a correction is shown in Table D which lists the results as obtained by the three methods. The values tabulated in row marked "Beam Method" have been computed by coefficients from Table A. In the next row called "Modified Beam Method," the effect of the horizontal displacement is included. The third row represents values computed according to ASCE Manual No. 31 using a single sinusoidal load. An examination of the M_ϕ values reveals that the unmodified beam method gives results at variance with the shell theory. While M_ϕ values recorded in Table D are small, and can be easily resisted by the concrete itself, in some cases serious underestimation of critical moments can occur. More disturbing is the fact that the moment could be of opposite sign. As seen in Table D much closer

agreement is obtained by the modified beam method. This good correlationship holds for all long barrel shells.

Another point which Table D illustrates is that an analysis by shell theory on the basis of a single sinusoidal load yields shear stresses that are too low. To avoid the necessity of applying additional loads, it is the general practice in many cases to multiply the shears obtained from a single sinusoidal load by 1.20. In this connection, the beam method implies that the distribution of shear at various cross sections is constant. In reality, near the supports the location of the point of maximum shear tends to be lower than at midspan. The magnitude of the shear stresses decreases somewhat above the neutral axis and increases below. With respect to reinforcing for shear stresses this change is not very signifi-

TABLE B — EFFECT OF A UNIT HORIZONTAL DISPLACEMENT

ϕ_k, deg	ϕ	T_ϕ (9)	M_ϕ (10)	ϕ_k, deg	ϕ	T_ϕ (9)	M_ϕ (10)
	ϕ_k	2463.0	62.82		ϕ_k	199.2	13.92
	$0.75\phi_k$	2451.1	50.96		$0.75\phi_k$	196.5	11.26
22.5	$0.50\phi_k$	2415.6	15.49	37.5	$0.50\phi_k$	188.6	3.35
	$0.25\phi_k$	2356.9	− 43.34		$0.25\phi_k$	175.7	− 9.60
	0	2275.5	−124.66		0	158.0	−27.24
	ϕ_k	1461.9	45.95		ϕ_k	145.5	11.53
	$0.75\phi_k$	1453.2	37.26		$0.75\phi_k$	143.3	9.32
25.0	$0.50\phi_k$	1427.2	11.30	40.0	$0.50\phi_k$	136.7	2.76
	$0.25\phi_k$	1384.3	− 31.63		$0.25\phi_k$	126.0	− 7.96
	0	1324.9	− 91.02		0	111.4	−22.50
	ϕ_k	912.9	34.65		ϕ_k	82.2	8.20
	$0.75\phi_k$	906.4	28.09		$0.75\phi_k$	80.7	6.62
27.5	$0.50\phi_k$	886.8	8.49	45.0	$0.50\phi_k$	76.0	1.94
	$0.25\phi_k$	854.4	− 23.86		$0.25\phi_k$	68.4	− 5.66
	0	809.8	− 68.50		0	58.2	−15.89
	ϕ_k	594.6	26.80		ϕ_k	49.6	6.06
	$0.75\phi_k$	589.5	21.71		$0.75\phi_k$	48.4	4.88
30.0	$0.50\phi_k$	574.3	6.54	50.0	$0.50\phi_k$	44.9	1.41
	$0.25\phi_k$	549.3	− 18.46		$0.25\phi_k$	39.3	− 4.19
	0	514.9	− 52.86		0	31.9	−11.65
	ϕ_k	401.2	21.17		ϕ_k	31.5	4.62
	$0.75\phi_k$	397.2	17.14		$0.75\phi_k$	30.6	3.72
32.5	$0.50\phi_k$	385.2	5.14	55.0	$0.50\phi_k$	27.9	1.06
	$0.25\phi_k$	365.4	− 14.59		$0.25\phi_k$	23.7	− 3.20
	0	338.4	− 41.66		0	18.1	− 8.81
	ϕ_k	279.0	17.03		ϕ_k	20.9	3.62
	$0.75\phi_k$	275.8	13.79		$0.75\phi_k$	20.2	2.91
35.0	$0.50\phi_k$	266.1	4.12	60.0	$0.50\phi_k$	18.1	0.82
	$0.25\phi_k$	250.3	− 11.74		$0.25\phi_k$	14.8	− 2.51
	0	228.6	− 33.43		0	10.5	− 6.84

cant and can be ignored in design. However, it does alter the magnitude of the transverse moment greatly.

In conventional shells, the depth of the supporting members is generally much greater than the shell. As a consequence of the difference in the stiffness between the two elements, the transverse moment M_ϕ at the support is zero. From this section, M_ϕ gradually increases to a maximum value. The rate of increase is a function of the chord to span ratio. Some insight on the longitudinal distribution of M_ϕ at the crown and

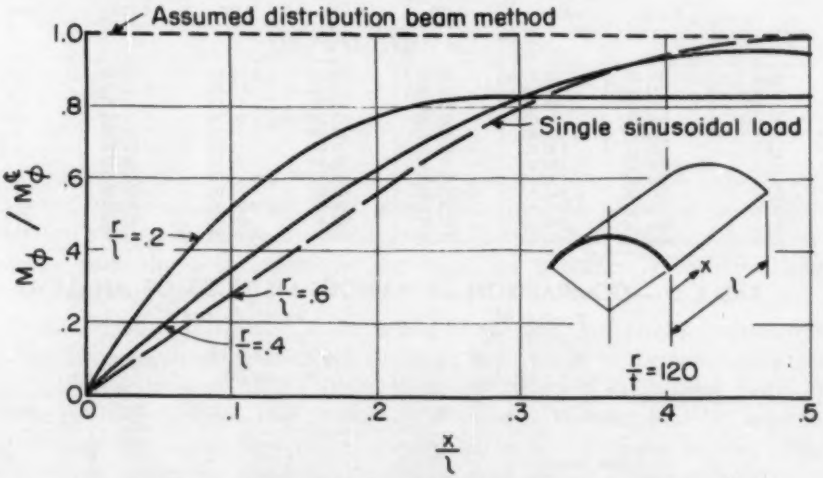


Fig. A — Longitudinal distribution of M_ϕ at crown

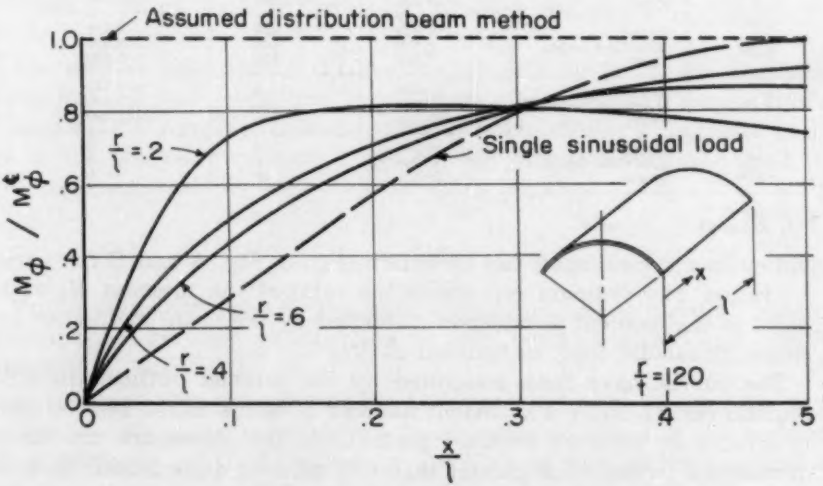
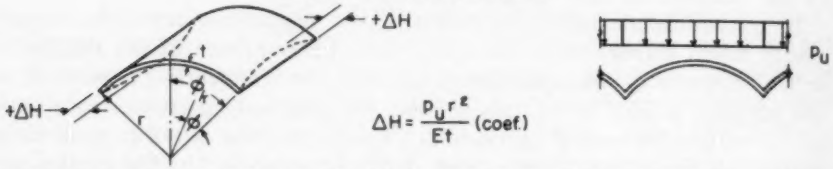


Fig. B — Longitudinal distribution of M_ϕ at valley

TABLE C — MEMBRANE HORIZONTAL DISPLACEMENT FOR UNIFORM LOADS



	r/l						
	0.100	0.125	0.150	0.175	0.200	0.225	0.250
ϕ_k, deg	$\Delta H/(P_u r^2/Et)$						
22.5	− 91.83	− 38.37	− 18.88	− 10.37	− 6.16	− 3.87	− 2.52
25.0	− 123.80	− 51.79	− 25.54	− 14.09	− 8.42	− 5.33	− 3.52
27.5	− 161.58	− 67.67	− 33.43	− 18.49	−11.09	− 7.07	− 4.71
30.0	− 205.26	− 86.02	− 42.55	− 23.59	−14.20	− 9.08	− 6.09
32.5	− 254.82	−106.85	− 52.91	− 29.38	−17.72	−11.38	− 7.66
35.0	− 310.10	−130.09	− 64.47	− 35.84	−21.66	−13.94	− 9.42
37.5	− 370.84	−155.63	− 77.17	− 42.94	−25.99	−16.77	−11.37
40.0	− 436.65	−183.31	− 90.94	− 50.65	−30.70	−19.83	−13.47
45.0	− 581.48	−244.21	−121.26	− 67.61	−41.05	−26.59	−18.13
50.0	− 739.50	−310.68	−154.34	− 86.14	−52.37	−33.98	−23.22
55.0	− 904.36	−380.03	−188.87	−105.47	−64.18	−41.70	−28.54
60.0	−1068.79	−449.20	−223.31	−124.77	−75.97	−49.41	−33.85

TABLE D — COMPARISON OF VARIOUS METHODS OF ANALYSIS

$t = 4$ in. $p_s = 25$ psf
$r = 10$ ft $p_d = 50$ psf
$l = 66.7$ ft $r/l = 0.15$
$\phi_k = 60$ deg $r/t = 30$

ϕ	Method	T_x*	T_ϕ*	S*	M_ϕ†
ϕ_k (Crown)	Beam	−29,750	− 980	0	−110
	Modified beam	−29,750	−1,190	0	−484
	Shell	−30,420	−1,460	0	−504
$0.75\phi_k$	Beam	−23,890	− 820	−4,370	− 40
	Modified beam	−23,890	−1,030	−4,370	−339
	Shell	−24,610	−1,260	−3,510	−347
$0.50\phi_k$	Beam	− 6,710	− 420	−6,910	90
	Modified beam	− 6,710	− 600	−6,910	6
	Shell	− 7,300	− 730	−5,600	25
$0.25\phi_k$	Beam	20,620	0	−5,940	82
	Modified beam	20,620	− 150	−5,940	341
	Shell	21,030	− 160	−4,860	357
0 (Valley)	Beam	56,240	110	0	−237
	Modified beam	56,240	0	0	470
	Shell	59,320	20	0	416

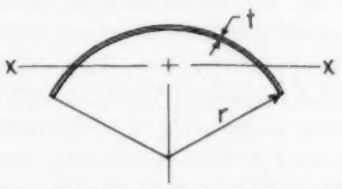
* lb per ft.
† ft-lb per ft.

valley in a typical shell can be obtained from Fig. A and B for various r/l ratios. The ordinate represents the ratio of the moment M_ϕ at any point to the moment at midspan computed by the beam method or by a single sinusoidal load, designated as M^s_ϕ.

The curves have been computed by the method outlined in ASCE Manual No. 31, using a sufficient number of terms in the Fourier series to achieve an accuracy within 1 percent. At the crown, the distribution of moment (when r/l is greater than 0.4) adheres quite closely to a sine distribution. Even for very long shells, the maximum moment is only

TABLE E — MOMENT OF INERTIA

$I_{x-x} = r^2t$ [Col. (11)]



ϕ_k, deg	(11)	ϕ_k, deg	(11)
22.5	0.00041	37.5	0.00502
25.0	0.00068	40.0	0.00687
27.5	0.00110	45.0	0.01216
30.0	0.00168	50.0	0.02017
32.5	0.00249	55.0	0.03174
35.0	0.00358	60.0	0.04782

reached at the quarter point. Along the valley, as one would expect, the moment increases at a slightly greater rate. In both places, it is evident that for values of r/l greater than 0.4, a sine distribution agrees more closely with the actual distribution than the constant value resulting from the beam method.

It should be noted that Tables A and B are only for shells continuous in the transverse direction. No attempt was made to provide data on single shells because of the infrequent use of such shells and because of the limited applicability of the beam theory. It must also be kept in mind that the excellent agreement shown in Dr. Chinn's article holds only when adjoining shells are identical and loaded symmetrically.

To facilitate the determination of the deflection, coefficients for determining the moment of inertia of the cross section are tabulated in Table E.

The writers concur with the statement made by Dr. Chinn that the beam method gives sufficient accuracy in many practical cases. However, a careful scrutiny, should always be made when r/l is greater than 0.6 or the boundary conditions and loading differs from those used in the examples.

By A. SIEV*

Prof. Chinn shows clearly the close agreement, on the bases of certain assumptions, between calculations by the beam method[1] and ASCE Manual No. 31.[2]

However, the statement that the beam method "can be used for any values of r/t and l/r, provided the assumptions involved in the method are sufficiently satisfied" should be qualified in view of the connection

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between the values of r/t, r/l, and the assumptions made. In the same context, the transverse stresses remain almost unchanged for any value of r/t and r/l provided they are in the same range.

This range can be determined by examination of the assumptions.

The first assumption is *theoretically* impossible. If all points on a transverse cross section are equally deflected vertically without horizontal displacement, deformation sets in — in contradiction to the beam method calculation. However, it can be proved that deformation of the cross section cannot exceed a certain amount (as M_ϕ has the role of distributing the load transversely) while the vertical deflection varies approximately as the fourth power of the shell span. Therefore, the higher the flexural rigidity compared to the longitudinal rigidity, the closer the agreement with Assumption (1); the same applies to Assumption (2). Factors influencing this relation are: r/l, r/t, ϕ_k, and continuity of shell; continuity over supports increases longitudinal rigidity, while transverse continuity (multiple barrel shell) increases shell arch rigidity.

Deflection according to theoretical Assumption (1) [Reference 1, p. 78] obeys Bernoulli's law of linear distribution of longitudinal stresses T_x.

Fig. 2^2 shows clearly how this takes place when l increases, and the effect of transverse continuity is seen by comparing Example 1^2 (curved T_x distribution) with Example 2 (almost linear T_x distribution). This shows that the condition $r/l < 0.2$ (Reference 2, p. 2) for straight line distribution of T_x in a single barrel shell is replaced by at least $0.9 > r/l$ in a multiple barrel shell (Fig. 8, Reference 5). All multiple barrel shells covered by the range $r/l < 0.6$ of Table 2^2, and more, (which includes almost all practical cases) are cases of straight line distribution. Let us refer to such a shell as a "long" shell.

In the numerical examples given by Prof. Chinn only such "long" shells are dealt with and therefore an almost straight line distribution of T_x was obtained. However, the four required values M_ϕ, S, T_x, T_ϕ [in view of Assumption (2)] are interrelated by three equations of equilibrium in both methods, so that if one of them is found by strain considerations the other three are determined unambiguously. T_x being almost the same by both methods, it is apparent that the other values must also be almost the same.

Had a "shorter" shell been studied (i.e., Example 1 of Reference 1) no such agreement would exist. These considerations lead to the suggestion of a new table for an inner shell (circular cross section), as a sequel to Prof. Chinn's work — based on functions of ϕ_k only, which will be much easier for use. The old tables[2] will continue to serve only for single shells or very short ones (Reference 2, Table 3). Definition of the range of long-span beams in the various conditions will be necessary in that case.

By ANTON TEDESKO*

The author is to be commended for a well prepared paper giving a simplified analysis of cylindrical shells. Since the range of applicability of simplified methods of design is often not fully realized by the reader of such and similar papers, it is the purpose of this discussion to point to the limitations in the use of approximate methods of design.

For many years it has been the practice of the company with which the writer is associated to devote considerable effort through field and office research toward gaining a better understanding of the behavior of shell structures. Our methods of design based on the theory of elasticity and on so-called exact solutions require fairly extensive calculations. We analyzed our solutions in an effort to see what approximations could be made to simplify calculations for future designs. From these observations the writer came to the conclusion that approximate methods of design are primarily useful for the purpose of preparing a preliminary estimate of materials or to determine variations of design values where an exact analysis of a similar structure is already in existence.

The writer has used an approximate approach similar to the method described in the author's paper and from past results knows that the total tensile and compressive forces checked rather well with those obtained by the exact solution, but that values for maximum direct stress and for transverse bending moments were unreliable, the exact values for bending moments sometimes being many times greater than those obtained by the approximate method.

For single radius cylindrical shells, the diagrams for direct stresses and moments are rather quickly computed by the exact method and it hardly seems worthwhile to save a small amount of time by following an approximate method leading to results which only the experienced designer can judge as to their accuracy.

The writer believes that the idea of reducing the shell design work and substituting approximate procedures can only be accepted where there is direction of the design group by an engineer versed and experienced in the behavior of shells. This experience can only be acquired through exact analyses of shells. Only by comparison with exact studies can a designer develop a feeling and judgment as to when, where, and within what ranges the results of simplified analyses may be useful and where departures from conventional details are permissible.

The author's paper has merit in the hands of the proper people and is considered a worthwhile contribution in the field of shell literature.

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By ALFRED ZWEIG*

Dr. Chinn has rendered a valuable service to the profession with his publication describing the cylindrical shell analysis simplified by the beam method. To analyze a shell as a beam is certainly much simpler than to apply the more general method contained in ASCE Manual No. 31.

Although there are definite limitations for the use of the beam method as emphasized by Dr. Chinn in the introduction to his paper, the example used in ASCE Manual No. 31 as well as the example in Pamphlet ST 37 of the Portland Cement Association are well within the range of this method. And the importance of Dr. Chinn's paper lies primarily in the fact that he has demonstrated how close the results of the approximate methods correspond to those of the more accurate analysis.

This paper is valuable also in that it furnishes the designer with a set of fully developed formulas pertaining to a rather commonly used shell shape.

There is, nevertheless, one great shortcoming in this paper which lessens its practical applicability to the every-day use by the designing engineer. Almost all values required for the shell design, such as moment of inertia, deformations, rotation, moments, normal force, etc., are represented as small differences of large quantities and, consequently, these values can only be obtained with any degree of accuracy by the use of a calculating machine exact to the seventh-decimal place.

Accuracy to this extent would require much more time and greater effort than the average engineer would be willing to invest — even for the design of a cylindrical shell roof; and, with its resultant susceptibility to innumerable mistakes, the practical value of this approach may be in doubt.

If the form in which Dr. Chinn presents his derivations were the only one available, the writer, for one, would prefer the tables of ASCE Manual No. 31, which permit the use of the slide rule, rather than a method which compels one to use seven-place decimals.

Fortunately, there is a way out of this dilemma. All formulas presented by Dr. Chinn can be developed in various trigonometric power series leading to expressions which avoid small differences of large quantities and which, therefore, are adaptable for ordinary slide rule use.

Further simplifications can be obtained by contracting certain terms. It is not, for instance, advisable first to determine the influence of the vertical loads at a far distant point from the origin, then to establish the moments due to the shear forces at the same point, and, finally to subtract one from the other value. It is, rather, preferable to combine the

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values of, say, Eq. (5) and (14) and (9) and (14) and to use the formula of the final difference.

In this connection, the choice of the statically indeterminate values by Dr. Chinn might be questioned. While it is true that his approach has the distinct advantage that the two statically indeterminates can be evaluated with two independent equations, each with only one unknown, the writer has used in a similar problem the three-hinged arch as the statically determinate main system. Although this leads to a set of two equations, one dependent upon the other, and the solution is, therefore, a bit more cumbersome than the elastic center approach, the writer prefers the three-hinged arch as the main system since the answers thus obtained are less sensitive to small errors. The reason for this lies in the general truth that any statically determinate main system should be chosen so that it comes as close as possible to the action of the indeterminate structure and that the redundants should add only minor corrections to the main system. From this point of view, the moments on the three-hinged arch will resemble the final moments much more than the moments of the statically determinate main system chosen by Dr. Chinn.

To be able to apply the power series approach to those equations which contain, in addition to the expression ϕ_k, also the value ϕ, such as, for instance, Eq. (9), (12), (14), etc., it is advantageous to express ϕ as fractions of ϕ_k.

Later in this discussion, all formulas presented by Dr. Chinn resulting in the difference of large values have first been transformed through power series and then properly combined into expressions which can be used with an ordinary slide rule. Formulas have been prepared for $\phi = 0$, $\phi = \phi_k/2$, and $\phi = \phi_k$.

There is another question which comes to mind in connection with Dr. Chinn's paper, and that is: What is the distribution of M_ϕ and T_ϕ values in the direction of the longitudinal span of the shell? Dr. Chinn does not discuss this question. ASCE Manual No. 31 recommends varying the M_ϕ and T_ϕ values according to sin $(\pi x)/l$. The recommendations of Manual No. 31, however, do not seem to be justified.

As pointed out by Dr. Chinn, the M_ϕ and T_ϕ values are solely a function of dVQ/Ib. For a uniformly distributed load dV/dx is constant for the full length of the beam and, consequently, the M_ϕ and T_ϕ values should remain constant for the full length of the shell and should not be reduced according to sin $(\pi x)/l$. The recommendations of Manual No. 31 in this respect are not on the safe side, and the designer should be warned not to reduce the reinforcing for M_ϕ and T_ϕ as suggested in Fig. 21, p. 54 of Manual No. 31.

The conclusions of the manual are understandable from the general approach therein. In the manual the uniformly distributed load is replaced by one following the cosine law in order to solve the differential

TABLE F — COEFFICIENTS K_n FOR POWER SERIES OF VARIOUS TRIGONOMETRIC FUNCTIONS

Function	ϕ^0	ϕ^1	$\frac{\phi^2}{2!}$	$\frac{\phi^3}{3!}$	$\frac{\phi^4}{4!}$	$\frac{\phi^5}{5!}$	$\frac{\phi^6}{6!}$	$\frac{\phi^7}{7!}$	$\frac{\phi^8}{8!}$	$\frac{\phi^9}{9!}$	$\frac{\phi^{10}}{10!}$	$\frac{\phi^{11}}{11!}$	$\frac{\phi^n}{n!}$
$\sin\phi$		1		−1		+1		−1		+1		−1	
$\cos\phi$	1		−1		+1		−1		+1		−1		
$\sin^2\phi$			2		−8		+32		−128		+512		2^{n-1}
$\cos^2\phi$	1		−2		+8		−32		+128		−512		2^{n-1}
$\sin^3\phi$				6		−60		+546		−4920		+44,286	$\frac{3}{4}(3^{n-1}-1)$
$\cos^3\phi$	1		−3		+21		−183		+1641		−14,763		$\frac{3}{4}(3^{n-1}+1)$
$\sin\phi\cos\phi$		1		−4		+16		−64		+256		−1024	2^{n-1}
$\sin^2\phi\cos^2\phi$			2		−32		+512		−8192		+131,072		2^{2n-3}
$\sin^3\phi\cos^2\phi$				6		−180		+4746		−120,840			$\frac{n!}{8}\sum_{m=1,3..}^{n-2}\frac{2^{2(n-m)}}{m!\,(n-m)!}$
$\sin^2\phi\cos^3\phi$			2		−44		+1022		−24,824				$\frac{n!}{8}\sum_{m=0,2,4..}^{n-2}\frac{2^{2(n-m)}}{m!\,(n-m)!}$
$\sin^4\phi\cos\phi$					24		−840		+23,184		−599,280		$\frac{n!}{8}\sum_{m=2,4..}^{n-2}\frac{(3^{n-m}-1)2^m}{m!\,(n-m)!}$
$\sin\phi\cos^4\phi$		1		−13		+241		−5293		+125,761			$\frac{3n!}{8}\sum_{m=0,2,4..}^{n-2}\frac{(3^{n-1}+1)2^{n-m}}{m!\,(n-m)!}$
$\sin\phi\cos^2\phi$		1		−7		+61		−547		+4921		−44,287	$1/4\,(3^n+1)$
$\sin^2\phi\cos\phi$			2		−20		+182		−1640		+14,762		$1/4\,(3^n-1)$

$$f(\sin\phi\ \cos\phi) = \Sigma K_n \frac{\phi^n}{n!}$$

Example: $\sin\phi\ \cos^2\phi - \phi - \frac{7\phi^3}{3!} - \frac{61\phi^5}{5!} - \frac{547\phi^7}{7!} - \frac{4921\phi^9}{9!} - \ldots$

equation. Consequently dV/dX follows the sine law from which the manual concludes that the M_ϕ and T_ϕ distribution should be the same.

The beam method, on the other hand, shows clearly that dV/dX is constant and that the recommendation of the manual in this respect must be properly adjusted.

It is true that, directly at the support, the M_ϕ and T_ϕ values will vanish because of the rigidity of the end supports; but the influence of the end support will disappear much faster than the sine law would indicate and, for practical purposes, the M_ϕ and T_ϕ values should be assumed to be more or less constant for the full length of the shell.

Finally, there should be mentioned a small omission in Dr. Chinn's paper, which otherwise was obviously done with greatest care and diligence. In the notations the definition of w was omitted. Upon inquiry by the writer, Dr. Chinn supplemented these notations, as follows:

$$\textit{For dead load: } w_{DL} = 2p_{DL}\, r\phi_k$$
$$\textit{For live load: } w_{LL} = 2p_{LL}\, r \sin\phi_k$$

DERIVATIONS

The following derivations are made with the use of the coefficients in Table F. The trigonometric series $\sin\phi = \phi - {}^3/3 + \phi^5\ 5 - \phi^7/7 + \ldots.$ and $\cos\phi = 1 - \phi^2/2 + \phi^4/4 - \phi^6/6 + \ldots.$ can be found in many handbooks. To properly transform all formulas pertaining to the design of cylindrical shells, the knowledge of the coefficients for the power series of other trigonometric functions is required. For this reason the writer has prepared Table F which contains the coefficients for these functions. Where feasible, a general expression for the coefficients is shown in the last column of the table. Since, in many equations, the coefficients with smaller exponents cancel out, indicating the fact that the function under consideration has a small value, it is necessary to develop some of the coefficients for expression up to and including the eleventh power of the series.

Eq. (1), (4), (8), and (17) through (21) of Dr. Chinn's paper do not require any modification. All other expressions have been developed in trigonometric series. Eq. (5a) and (9a) have been added. They are the result of combining Eq. (14) with Eq. (5) and (9), respectively. To do so, the expression for the moment of inertia in Eq. (2) has been substituted in Eq. (14), and the value for w has been inserted either as $2pr\phi_k$, when considering dead load [Eq. (5a)], or as $2pr\sin\phi_k$ for live load [Eq. (9a)]. Eq. (5a) and (9a) are especially advantageous when considering the moments for $\phi = 0$, since the individual application of Eq. (5), (9), and (14) may lead to results not accurate enough when restricting the design to the use of the slide rule.

Omitting the tedious derivations, the final results are:

$$\textit{Eq. (2):} \qquad I = \frac{2r^3}{315}\,(7\phi_k^5 - \phi_k^7)$$

$$\textit{Eq. (3):} \qquad \phi = 0 : Q = 0$$

$$\phi = \frac{\phi_k}{2} : Q = \frac{r^2}{8}\left(\phi_k^3 - \frac{\phi_k^5}{16}\right)$$

$$\phi = \phi_k : Q = 0$$

Dead load

$$\textit{Eq. (5):} \qquad \phi = 0 : M = \frac{pr^2}{2}\left(\phi_k^2 - \frac{\phi_k^4}{12}\right)$$

$$\phi = \frac{\phi_k}{2} : M = \frac{pr^2}{8}\left(\phi_k^2 - \frac{11}{48}\,\phi_k^4\right)$$

$$\phi = \phi_k : M = 0$$

Eq. (5a) [combining Eq. (5) with Eq. (14)]:

$$\phi = 0 : \Sigma M = \frac{252\phi_k^7 - 103\phi_k^9}{192\,(7\phi_k^5 - \phi_k^7)}\,pr^2$$

$$\phi = \frac{\phi_k}{2} : \Sigma M = \frac{11{,}088\phi_k^7 - 5031\phi_k^9}{16{,}384\,(7\phi_k^5 - \phi_k^7)}\,pr^2$$

$$\phi = \phi_k : \Sigma M = 0$$

$$\textit{Eq. (6):} \quad \frac{Et^3}{12}\,\theta_{EC} = \frac{pr^3}{2}\left(\frac{\phi_k^3}{3} - \frac{\phi_k^5}{20}\right)$$

$$\textit{Eq. (7):} \qquad \frac{Et^3}{12}\,(\triangle_k)_{EC} = \frac{pr^4}{360}\left(7\,\phi_k^5 - \frac{17}{14}\,\phi_k^7\right)$$

Uniformly distributed load

$$\textit{Eq. (9):} \qquad \phi = 0 : M = \frac{pr^2}{2}\left(\phi_k^2 - \frac{\phi_k^4}{4}\right)$$

$$\phi = \frac{\phi_k}{2} : M = \frac{pr^2}{8}\left(\phi_k^2 - \frac{7}{12}\,\phi_k^4\right)$$

$$\phi = \phi_k : M = 0$$

Eq. (9a) [combining Eq. (9) with Eq. (14)]:

$$\phi = 0 : \Sigma M = \frac{84\phi_k^7 - 67\phi_k^9}{64\,(7\phi_k^5 - \phi_k^7)}\,pr^2$$

$$\phi = \frac{\phi_k}{2} : \Sigma M = \frac{11{,}088\phi_k^7 - 9567\phi_k^9}{16{,}384\,(7\phi_k^5 - \phi_k^7)}\,pr^2$$

$$\phi = \phi_k : \Sigma M = 0$$

$$\textit{Eq. (10):} \qquad \frac{Et^3}{12}\,\theta_{EC} = \frac{pr^3}{2}\left(\frac{\phi_k^3}{3} - \frac{3}{20}\,\phi_k^5\right)$$

$$Eq.\ (11): \quad \frac{Et^3}{12}(\triangle_h)_{EC} = \frac{pr^4}{180}\left(\frac{7\phi_k^5}{2} - \frac{11\phi_k^7}{7}\right)$$

Shear difference

$$Eq.\ (12): \quad \phi = 0 : F_v = \frac{wr^3}{45I}\left(\phi_k^3 - \frac{\phi_k^7}{7}\right)$$

$$\phi = \frac{\phi_k}{2} : F_v = \frac{wr^3}{320I}\left(\frac{47\phi_k^5}{9} - \frac{2869}{896}\phi_k^7\right)$$

$$\phi = \phi_k : F_v = 0$$

$$Eq.\ (13): \quad \phi = 0 : F_h = \frac{wr^2}{24I}\left(-\phi_k^4 + \frac{7}{30}\phi_k^6\right)$$

$$\phi = \frac{\phi_k}{2} : F_h = \frac{3wr^2}{128I}\left(-\phi_k^4 + \frac{13}{40}\phi_k^6\right)$$

$$\phi = \phi_k : F_h = 0$$

$$Eq.\ (14): \quad \phi = 0 : M = \frac{wr^4}{144I}\left(\phi_k^6 - \frac{7}{60}\phi_k^{11}\right)$$

$$\phi = \frac{\phi_k}{2} : M = \frac{wr^4}{46{,}080I}\left(29\phi_k^6 - \frac{907}{336}\phi_k^{11}\right)$$

$$\phi = \phi_k : M = 0$$

$$Eq.\ (15): \quad \frac{Et^3}{12}\theta_{EC} = \frac{wr^5}{630I}\left(\phi_k^7 - \frac{19}{180}\phi_k^9\right)$$

$$Eq.\ (16): \quad \frac{Et^3}{12}(\triangle_h)_{EC} = \frac{wr^6}{648I}\left(\frac{\phi_k^9}{7} - \frac{271\phi_k^{11}}{11{,}550}\right)$$

NUMERICAL EXAMPLE

The application of these formulas will be demonstrated on Example 2 from ASCE Manual No. 31 which was also used by Dr. Chinn. Note, however, how much simpler the arithmetic is and that in no instance more than slide rule accuracy is required to get results sufficiently close to the more exact values of Manual No. 31. The figures in bracket are, unless otherwise mentioned, those obtained by Dr. Chinn.

The following basic values will be used:

$\phi_k = 40\ \text{deg} = 0.698$	$\phi_k^8 = 0.0564$	$r^4 = 924{,}000$
$\phi_k^2 = 0.487$	$\phi_k^9 = 0.0394$	$r^5 = 28{,}600{,}000$
$\phi_k^3 = 0.340$	$\phi_k^{10} = 0.0275$	$r^6 = 888{,}000{,}000$
$\phi_k^4 = 0.237$	$\phi_k^{11} = 0.0192$	$\sin\phi_k = 0.643$
$\phi_k^5 = 0.166$	$r = 31$ ft	$\frac{\sin\phi_k}{\phi_k} = 0.921$
$\phi_k^6 = 0.116$	$r^2 = 961$	
$\phi_k^7 = 0.0808$	$r^3 = 29{,}800$	

Dead load $= p = 47$ psf
Live load $= p = 25$ psf

$$w_{DL} = 2p_{DL}\, r\phi_k = 2 \times 47 \times 31 \times 0.698 = 2033 \text{ lb per ft}$$
$$w_{LL} = 2p_{LL}\, r \sin\phi_k = 2 \times 25 \times 31 \times 0.643 = 997 \text{ lb per ft}$$
$$w = 3030 \text{ lb per ft}$$

From Eq. (2):

$$I = \frac{2 \times 29{,}800}{315}(1.162 - 0.081) = 205 \text{ ft}^2 \quad [204.777]$$

From Eq. (6):

$$\frac{Et^2}{12}\theta_{EC} = \frac{47 \times 29{,}800}{2}(0.1133 - 0.0083) = 73{,}700 \quad [73{,}710.36]$$

From Eq. (10):

$$\frac{Et^2}{12}\theta_{EC} = \frac{25 \times 29{,}800}{2}(0.1133 - 0.0249) = 32{,}800 \quad [33{,}718.87]$$

From Eq. (15):

$$\frac{Et^2}{12}\theta_{EC} = \frac{3030 \times 28{,}600{,}000}{630 \times 205}(0.0808 - 0.0041) = 51{,}500 \quad [51{,}685.93]$$

From Eq. (20):

$$\frac{Et^2}{12}\theta_{EC} = 31 \times 0.698\, M_{EC} = 21.6\, M_{EC} \quad [21.642092\, M_{EC}]$$

Solving for M_{EC}:

$$73{,}700 + 32{,}800 - 51{,}500 = 21.6\, M_{EC}$$
$$M_{EC}\,\frac{55{,}000}{21.6} = 2550 \text{ ft-lb per ft} \quad [2575.69]$$

From Eq. (7):

$$Et^2(\triangle_h)_{EC} = \frac{47 \times 924{,}000}{360}(1.162 - 0.098) = 128{,}000 \quad [128{,}554.03]$$

From Eq. (11):

$$Et^2(\triangle_h)_{EC} = \frac{25 \times 924{,}000}{180}(0.581 - 0.127) = 58{,}100 \quad [59{,}552.1]$$

From Eq. (16):

$$Et^2(\triangle_h)_{EC} = \frac{3030 \times 880{,}000{,}000}{648 \times 205}(0.00563 - 0.00045) = 104{,}000 \quad [106{,}379.81]$$

From Eq. (19):

$$Et^2(\triangle_h)_{EC} = \frac{205}{2}H_{EC} = 102.5\, H_{EC} \quad [102.389\, H_{EC}]$$

Solving for H_{EC}:

$$128{,}000 + 58{,}100 - 104{,}000 = 102.5\, H_{EC}$$
$$H_{EC} = \frac{82{,}500}{102.5} = 800 \text{ lb per ft} \quad [798.19]$$

Before getting the final moments on the statically indeterminate system, the moments M_o on the statically determinate systems resulting from the vertical loads and the influence of the shear difference are obtained for $\phi - \phi_2/2$ with the help of Eq. (5), (9), and (14) and for $\phi = 0$ with Eq. (5a) and (9a).

For $\phi = \phi_2$: $M_o = 0$

For $\phi = \frac{\phi_2}{2}$:
$$M_o = \frac{-47 \times 961}{8}(0.487 - 0.054)\ \frac{-25 \times 961}{8}(0.437 - 0.138) + \frac{3020 \times 924{,}000}{46{,}080 \times 205}(3.37 - 0.152)$$
$$= -2450 - 1050 + 950 = -2550 \text{ ft-lb}$$

For $\phi = 0$:
$$M_o = -47 \times 961 \frac{20.36 - 4.06}{192 \times 1.081} - 25 \times 961 \frac{6.78 - 2.64}{64 \times 1.081}$$
$$= -3540 - 1430 = 4970 \text{ ft-lb}$$

Moments $\bar{M}$ reflecting the influence of the statical indeterminacy are:

For $\phi = \phi_2$ $\bar{M} = 2550 + 800 \times 31\ (0.766 - 0.921) = -1290$ ft-lb

For $\phi = \frac{\phi_2}{2}$ $\bar{M} = 2550 + 800 \times 31\ (0.940 - 0.921) = +3020$ ft-lb

For $\phi = 0$ $\bar{M} = 2550 + 800 \times 31\ (1.000 - 0.921) = +4520$ ft-lb

The final moments are established by adding M_o to $\bar{M}$. The values in brackets are the moments derived on p. 56 of ASCE Manual No. 31 and multiplied with the coefficient $\pi/4$.

For $\phi = \phi_2$ $M = 0 - 1290 = -1290$ ft-lb [−1170]

$\phi = \frac{\phi_2}{2}$ $M = -2550 + 3020 = +470$ ft-lb [+394]

$\phi = 0$ $M = -4970 + 4520 = -450$ ft-lb [−435]

This comparison shows that the answers obtained with a slide rule are sufficiently close to those resulting from the more accurate analysis of Manual No. 31.

AUTHOR'S CLOSURE

The writer is indebted to all the discussers for their excellent comments on his paper. Their discussions have increased the value of the original paper manyfold.

Messrs. Siev and Parme and Conner point out that the assumption of equal vertical deflections of all points and no horizontal deflections is theoretically impossible. They recognize, however, that this assumption need not be exactly satisfied, a point the writer intended to be expressed in the statement "provided the assumptions involved in the method are *sufficiently* satisfied."

Messrs. Siev, Parme and Conner, and Tedesko have commented on the range of applicability of the beam method. Mr. Siev suggests that the method is applicable for an r/l of at least 0.9, while Messrs. Parme and Conner call for a "careful scrutiny" of its applicability when r/l exceeds 0.6. The two shells shown in the writer's paper had r/l of 0.5 and 0.9, respectively. This can be used as a guide; however, it should be obvious that r/l is not the sole parameter to be used in setting a limit of applicability. r/t and ϕ_k would certainly be expected to have an affect. Until someone makes a sufficient number of comparative designs between the beam method and the exact method, the limits of applicability cannot be defined. The writer is of the opinion that competent designers possess sufficient judgment to decide when the assumptions of the beam method are sufficiently satisfied.

The intent of the writer's original paper was to present formulas applicable to one particular structure, the cylindrical shell of constant thickness. Mr. Ketchum has ably shown how the principles of the beam method can be applied to other shells. He also points out that the beam method gives a good insight on the physical action that takes place in shell structures in terms that are familiar to the average designer.

The writer is deeply indebted to Messrs. Parme and Conner for their extensive work in preparing the tables of coefficients which greatly simplify the calculations involved in the beam method. These tables are essentially the ones Mr. Siev has suggested should be prepared. The calling attention to the error induced by membrane deflections and the preparation of tables for calculating this effect are very much appreciated. Many readers would no doubt be interested in seeing the equations used to prepare these corrections, and it is hoped that Messrs. Parme and Conner will publish them. Perhaps this correction eliminates some of Mr. Tedesko's objections to the beam method.

Mr. Zweig's contribution is a worthy one. The writer had thought about trying to simplify his equations; but when a solution by these equations did not agree with one using formulas from Reference 1 which appeared to be a power series solution, it was erroneously concluded that a power series solution was not accurate. Fortunately Mr. Zweig did not reach the same conclusion.

Messrs. Parme and Conner as well as Mr. Zweig re-emphasize the writer's statement that his equations require a high degree of accuracy in numerical calculation and are therefore not suited to slide rule calculation. Whether one would use the writer's equations in preference to those of ASCE Manual No. 31 is a matter of individual choice. It should be pointed out, however, that Manual 31 limits itself to r/t of 100 and 200 whereas the writer's equations contain no such limitation. It is doubtful that Mr. Zweig would choose the equations of Manual 31 for other values of r/t in preference to the writer's equations. Mr. Zweig's suggestion of using the three-hinged arch as the statical arch structure is

an excellent one, and coupled with the method of column analogy would not require the solution of simultaneous equations.

Messrs. Zweig and Parme and Conner have discussed the matter of the longitudinal distribution of M_ϕ. The exact solution indicates that M_ϕ begins to decrease at a significant distance from the supports. It should be remembered that the exact solution is based on supports which are basically diaphragms and includes the effects of twisting moments.

It seems to the writer that the matter of twisting moments has been mishandled in structural practice. In both plate and shell structures, solutions are obtained assuming that twisting moments do exist (sometimes reducing bending moments by as much as 50 percent), but the shearing stresses which would be induced by these twisting moments are never checked to see if they are excessive. In reinforced concrete there is some question as to whether plates and shells are capable of resisting moments. The writer recalls hearing of tests performed in Sweden that indicated slabs are incapable of resisting twisting moments after they crack. All the shells with which the writer is familiar were subject to considerable shrinkage cracking.

Another practical aspect to cylindrical shells is the matter of the actual loading versus the assumed loading. The live load on a shell is normally snow load assumed to be uniform on a horizontal projection. More likely the snow will tend to be deeper in the valleys than at the crown. The effect of uniform-load membrane deflection on M_ϕ as compared to negligible effect for dead load would indicate that the actual distribution of load has great influence on M_ϕ.

Discussion of a paper by M. F. Kaplan:

Flexural and Compressive Strength of Concrete as Affected by the Properties of Coarse Aggregates*

By J. F. McLAUGHLIN, B. G. SINGH, and STANTON WALKER and DELMAR L. BLOEM

By J. F. McLAUGHLIN†

Dr. Kaplan has made an interesting contribution to the literature concerned with variations in the flexural and compressive strength of concrete. There are, however, several questions that should be raised in connection with the statistical analysis of the data. This discussion is confined to these matters.

Table A shows the general experimental layout used by Dr. Kaplan. It is an expansion of Table 6 to include space for the individual observations that go to make up the means shown in his Table 6. A similar layout was used for flexural strength. If we assume that the specimens made on different days are replicates and not a nested effect, this is a three-way classification with three observations per cell. If all interactions are considered possible, the mathematical model is

$$Y_{ijkl} = \mu + A_i + M_j + AM_{ij} + C_k + AC_{ik} + MC_{jk} + AMC_{ijk} + \varepsilon_{ijkl}$$

where A = aggregate effect, M = mixture or gradation effect, C = effect of curing time, and ε = random effect.

Since the purpose of this investigation was to "... determine the effects of the properties of 13 different coarse aggregates on the flexural and compressive strength of concrete ...", it would appear that the first step should be an analysis of variance of the data so that it can be determined if aggregate effects are significant. The analysis and appropriate F-test follow from the model. If it is found that the means are significantly different (at say the 5 percent level), one can then proceed to further analysis which might be to make individual comparisons of the means in question by the Duncan procedure followed perhaps, by multiple regression. If, on the other hand, these

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TABLE A—EXPERIMENTAL LAYOUT OF COMPRESSIVE STRENGTH, PSI

Aggregate		Mixture or aggregate grading								
		I			II			III		
		7 days	28 day	91 days	7 days	28 days	91 days	7 days	28 days	91 days
A	Batch 1									
	Batch 2									
	Batch 3									
	Mean	7780	9770	10,360	3770	5910	6880	1880	3140	4010
B	Batch 1									
	Batch 2									
	Batch 3									
	Mean	8080	10,560	11,640	4100	6240	7380	2140	3330	4140
N	Batch 1									
	Batch 2									
	Batch 3									
	Mean	8140	10,410	11,460	4250	6500	7500	2170	3340	4070

means are not significantly different by our test criterion, then we must conclude that chance variation alone can account for the observed differences and perhaps this is where the analysis should end.

There is no statement in Dr. Kaplan's paper to indicate that such a significance test was made and since only averages are reported, one does not have sufficient information available to make the analysis himself. This leaves the reader uncertain as to the validity of the subsequent conclusions that are drawn, especially when he is aware that it is not unusual to find that the results of strength tests on concrete have a coefficient of variation of 5 percent.

With respect to the regression analysis that was performed, the results of which are shown in the author's Table 9, it is indicated that the coefficients for shape and surface texture were not significant in the majority of cases. Also, there does not appear to be a good theoretical basis for obtaining the R^2, reported as 84 percent for flexural strength, by averaging the individual values found in the separate regression analyses. It would help the reader if the individual values of R^2 were reported together with a statement as to whether these analyses were performed on means or individual observations.

In summary, the paper leaves much unsaid and leaves much to conjecture on the part of the reader. Since strong statements are made regarding the influence of aggregate characteristics on flexural and compressive strength of concrete, some of these points need clarification.

By B. G. SINGH*

We are indebted to Dr. Kaplan for his interesting paper. The absence of more specific conclusions from such an obviously painstaking study should not detract from its usefulness. The data presented may well serve as a foundation for future investigations in a field which seems promising. However, it would seem possible to give a somewhat different interpretation than that given.

Although Eq. (1) and (2) clearly indicate that strength, all other factors being equal, is mainly influenced by:

(1) The modulus of elasticity of the coarse aggregate, e.
(2) The angularity number of the coarse aggregate, a.
(3) The surface texture of the coarse aggregate, s.

There are obviously other factors not taken into account in view of the poor correlation between the equations and the results at any given age. Even after averaging the constants and results for various ages the correlation drawn in Fig. 6 still shows reasonable scatter. Dr. Kaplan makes three suggestions in an attempt to explain the results:

(1) The rougher surface as measured by s will result in greater adhesive force between the cement and the aggregate.
(2) The angularity as measured by a contributes to strength because angularity is associated with greater surface area.
(3) Lower modulus of elasticity produces lower strength because it deforms easier.

While a rough surface will undoubtedly contribute to mechanical bond, the development of adhesion is a function of the physico-chemical properties of the surface and the adherend. Although there is no known method of measuring such surface properties, in relation to bond, these could be inferred qualitatively from test results of the type obtained. The suggestion that the more angular material with its greater surface area contributes to strength seems to be somewhat doubtful for an increase in surface area will lead to a decrease in cement per unit aggregate area and so lead to lower strength. It might be inferred from the suggestion that if the coarse aggregates were broken down further to increase the surface area a higher strength would have been obtained, which is unlikely. Interlocking of the coarse aggregate due to roughness and angularity to give higher strength is also doubtful, for the lean mix is less sensitive to aggregate type.

As the modulus of elasticity of the aggregate is many times greater than that of the resultant concrete, it is clear that the cement matrix has a lower value. Also as the mean diameter of the aggregate particle decreases, relatively more cement will be associated with it and as a result the regions of fines will deform more readily and so tend to cause

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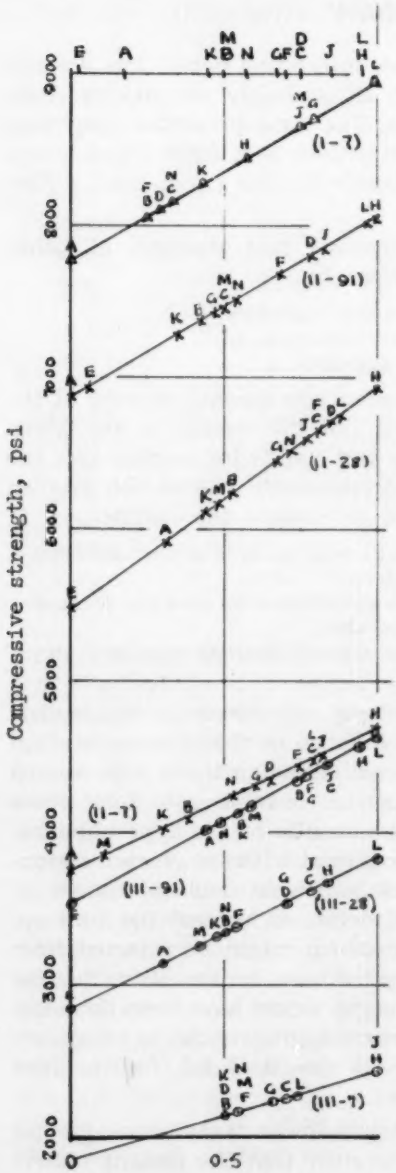

Fig. A—Compressive strength plotted with respect to highest and lowest values against an arbitrary scale to show the average position of each aggregate

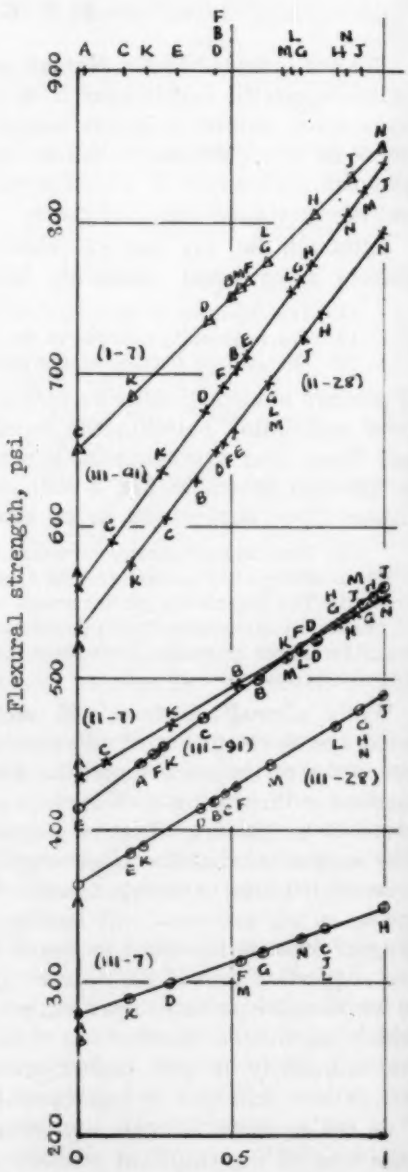

Fig. B—Flexural strength plotted with respect to highest and lowest values against an arbitrary scale to show the average position of each aggregate

differential movement within the mass — particularly when highly stressed. The larger particles such as the coarse aggregate will deform less readily and so develop higher bond stress, in some respects similar to reinforced concrete. Thus the larger the aggregate size and the higher the modulus of elasticity, the higher will tend to be bond stress, and for a given bond strength a higher e value should give a lower strength. This reasoning leads to the conclusion that type of surface becomes more important as the size of the aggregate increases, which may explain the significant effect of the coarse aggregate in spite of the fact that it contributed only between 8 to 16 percent of the total aggregate surface.

The modulus of elasticity, angularity, and surface texture must be related to the type and size of crystals forming the material. As the type and size of crystals will also determine the kind of mechanical and physico-chemical bond between the particles and the cement matrix, it would appear that the factors shown to be related to strength in the present paper are inseparable from those affecting bond. If bond is assumed to influence strength as outlined, then it is conceivable that although Eq. (1) and (2) may predict the same strength for two aggregates, significantly different results could be obtained, which may explain some of the scatter obtained.

It has been suggested in an earlier discussion that strength of concrete may be considered to be closely related to: (1) water-cement ratio; (2) bond between cement matrix and aggregate particles; (3) strength of aggregate particles; and (4) voids.

The water-cement ratio determines the strength of the cement matrix and influences both strength of the matrix and bond; and bond stresses develop provided that the matrix has sufficient strength and the aggregates are stressed to crushing point. Thus at low strengths, water-cement ratios tend to determine point of failure. Thus the effect of aggregate type is of less significance with lean mixes and high water-cement ratios, whereas at higher concrete strengths, bond and aggregate strength assume greater importance.

If compressive strength for each age of test and mix proportion is plotted against an arbitrary scale, say 0 to 1, 0 representing the lowest and 1 representing the highest and the other remaining 11 results fitted in their relative position with respect to strength (see Fig. A), performance of each could be read off on the arbitrary scale. It will be observed that except for the behavior of Mix 1 for 28 and 91 days the aggregates performed with surprising consistency, with H and L occupying approximately the highest average position and E occupying the lowest. Similar plots (see Fig. B) for flexual strength show J, N, and H occupied the highest average position whereas A occupied the lowest. It will be observed that N performed significantly better with

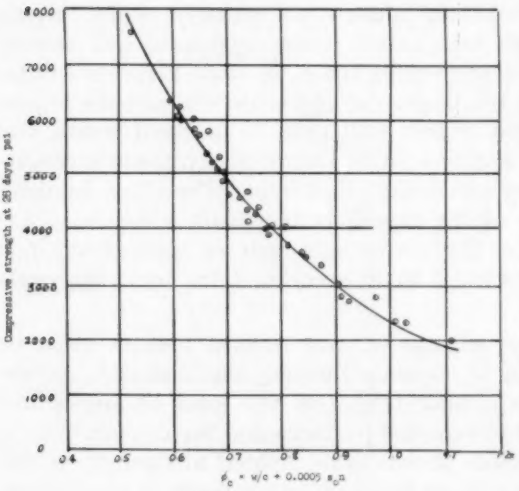

Fig. C — Compressive strength plotted against ϕ_c for aggregate of specific surface varying between 22 and 68 sq cm per g and $n = 6$

respect to flexual strength than compressive strength. Thus it would seem that a coarse aggregate that gives a high flexural strength may not necessarily give a high compressive strength as indicated by Eq. (2).

It has been shown* (Fig. C) that concrete strength for a given aggregate-cement ratio depended on both water-cement ratio and specific surface of the aggregate and a single curve for Ham River aggregates was obtained when strength was plotted against:

$$\phi_c = w/c + kS_o n$$

Where w/c = water-cement ratio by weight
S_o = specific surface of the aggregate, sq cm per g
n = aggregate-cement ratio by weight
k = a constant: 5×10^{-4} for compressive strength and 1×10^{-3} for flexural strength

It has been suggested that the constant k may depend on the properties of the aggregate and possibly also on the properties of the cement. Thus for given w/c, S_o, and n, strength may be reduced by a higher k value which could be interpreted as due to poorer bond.

If the aggregate giving the highest strength is assumed to give a perfect bond and strength as assumed to be determined by water-cement ratio alone, i.e., $k = 0$, and the relationship between strength and ϕ_c are similar to that shown in Fig. C except for a displacement of the curve, then it is possible to obtain k values for the average position occupied by each aggregate, if the specific surfaces of the aggregates are known. Assuming that the fine aggregates were similar to Ham River it is pos-

*Singh, B. G., "Specific Surface of Aggregates Related to Compressive and Flexural Strength of Concrete," ACI JOURNAL, V. 29, No. 10, Apr. 1958 (*Proceedings* V. 54), pp. 897-907; discussion: V. 30, No. 6, Part 2, Dec. 1958 (*Proceedings* V. 54), pp. 1373-1381.

sible to estimate the specific surface of each of the three gradings. With this assumption, specific surface of Grading 1 was estimated to be 12.7 sq cm per g; Grading 2, 16.7 sq cm per g; and Grading 3, 22.5 sq cm per g with a variation of about ± 6 percent due to shape and specific gravity of the various coarse aggregate. Thus assuming $k = 0$ for H and L, the following values $\times 10^{-4}$ were obtained for compressive strength:

A	B	C	D	E	F	G	H	J	K	L	M	N
6.05	3.55	1.45	1.45	7.2	1.94	2.16	0	0.8	4.04	0	3.55	2.65

These values could be used to relate ϕ_c to strength (see Fig. D) for each mix proportion and age of test, except for Mix 1 for 28 and 91 days, which compares favorably with Fig. 6.

Similarly a set of values could be obtained for flexural strength assuming $k = 0$ for J. The following values $\times 10^{-4}$ were obtained:

A	B	C	D	E	F	G	H	J	K	L	M	N
19.0	10.0	17.0	10.0	13.0	10.0	4.4	1.6	0	15.0	4.8	5.2	1.0

These, as for compressive strength, were used to calculated ϕ_r and plotted against flexural strength (Fig. E). If the results included a greater number of water-cement and aggregate-cement ratios it would have been possible to draw up a single relationship for each age of test for both compressive and flexural strength.

An obvious criticism of the values obtained for k is that it is based on the specific surface of the combined aggregate to which the coarse aggregate contributed a small part. However, as there is reason to believe that the surface properties of the coarse are of greater significance,

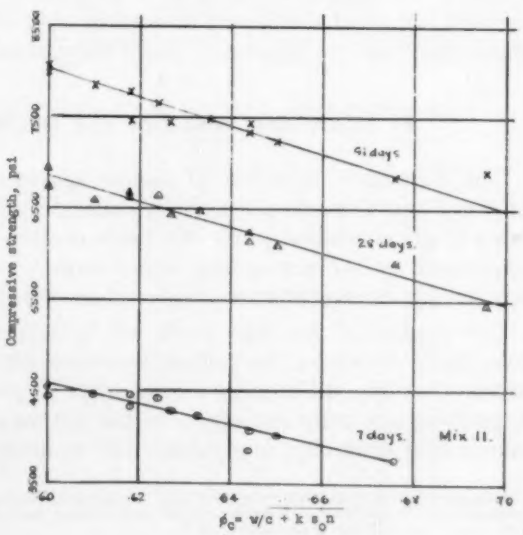

Fig. D — Correlation between compressive strength and ϕ_c for various aggregates used in study

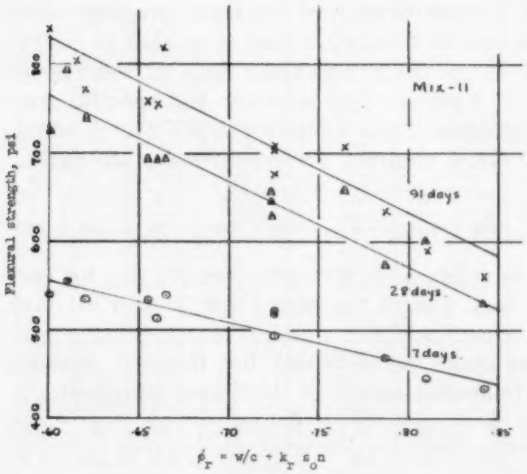

Fig. E — Correlation between flexural strength and ϕ_c for various aggregates used in study

this criticism should not be of over-riding importance. It might be noted that this is also a criticism of Dr. Kaplan's equations.

The effect of the various size groups should be of great interest, but proper assessment must depend on future work.

The great advantage in viewing the problem according to the concepts outlined is that there is no need to measure *e*, *a*, and *s* as tests could be made with the aggregates in concrete mixes under standard conditions and the decrease in strength resulting from the surface and other properties of the aggregate could be evaluated in terms of equivalent water-cement ratio and the saving in cement that will result with the use of a particular type of aggregate could be assessed.

By STANTON WALKER and DELMAR L. BLOEM*

The discussion of effect of coarse aggregate properties on the compressive and flexural strength of concrete presented by Dr. Kaplan has been of great interest to us. We have studied the data carefully. We are impressed by the meticulous detail with which the author has carried out the laborious statistical analyses on which his conclusions are based.

Our studies of the data have led to somewhat different conclusions than those drawn by the author. We have not been able to convince ourselves that the differences in strength of concrete made with the 13 aggregates are truly explained by the author's Eq. (1) and (2) involving modulus of elasticity, angularity, and roughness of aggregate. It is true

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that these equations appear to fit the data very well, but to say that they portray their significance may be misleading.

The concretes, made with aggregates of widely varying characteristics, all had the same proportions by weight and the same water-cement ratio for each of the three basic mixes. The proportions took no account of differences in specific gravity of the coarse aggregates; consequently, the quantity of cement per unit of volume of concrete varied. Further, for each basic mix, the ratio of fine to coarse aggregate by weight was the same throughout, taking no account of differences in particle shape, void content, or solid volume of the coarse aggregate. Accepted proportioning methods would require the ratios of fine to coarse aggregate to reflect coarse aggregate void content.

Certainly, if the mixtures with high-void-content coarse aggregate contained enough fine aggregate, then those with low-void-content aggregate were over-sanded. That the use of fixed proportions would produce concretes differing considerably in slump should also be expected. This effect is indicated in Table 3 where compacting factors suggest differences in slump of as much as 3 or 4 in.

The use of a constant water-cement ratio for each basic mix is not sufficient, in our opinion, to make the concretes comparable. The cement contents differed; the ratios of fine to coarse aggregate were not comparable; and the consistencies were not the same. These factors likely may have been principally responsible for the differences in strength which were found.

As a matter of fact, the variations in strength were not enough to leave much room for influence by any of the variables. For compressive strength, they were not substantially greater than might have been expected if only one aggregate had been tested 13 times. For the flexural

TABLE B—COEFFICIENTS OF VARIATIONS FOR 13 CONCRETES*

Mix	Age at test, days	Coefficients of variation, percent	
		Flexural strength	Compressive strength
I (1:3.08)	7	7.8	4.8
	28	10.2	5.5
	91	9.7	6.0
II (1:7.53)	7	7.4	5.9
	28	10.2	6.0
	91	11.6	4.7
III (1:10.25)	7	5.4	6.2
	28	9.1	7.2
	91	9.2	6.5

*These coefficients are calculated for the 13 values, representing the 13 aggregates shown in Table 5.

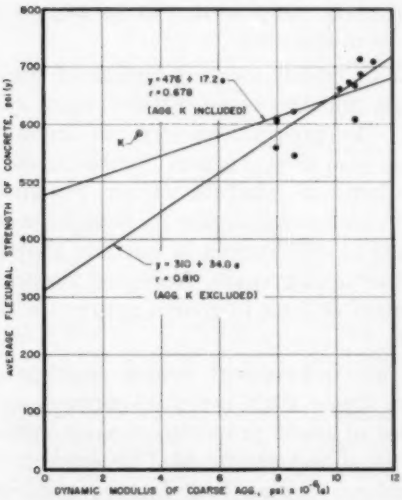

Fig. F—Relationship of concrete flexural strength to modulus of elasticity of coarse aggregate

tests, the variations certainly were not more than twice what would have resulted from testing one aggregate 13 times. This is demonstrated by the coefficients of variation, calculated for each age and each mix, shown in Table B.

Since flexural strengths varied substantially more than compressive, we shall restrict our discussion to them. Further, we shall deal with average strengths instead of, as the author did, with individual strengths and average relationships. It can be shown that either procedure gives about the same results. Table C summarizes the principal data on which our discussion is based.

Of the three characteristics which the author considered to correlate well with strength, the dynamic modulus of aggregate was said to exert the most influence. Two regression lines for dynamic modulus versus flexural strength are shown in Fig. F. These differ only in that, in one case, the out-of-character point for Aggregate K (with modulus of elasticity of only 3.3) has been omitted. Including K, the correlation coefficient is 0.678 which is significant with a 2 percent probability of error. That is to say, there are only about two chances in 100 that the rela-

TABLE C—SUMMARY OF DATA ON AGGREGATES AND CONCRETE*

Aggregate identification	Aggregate				Concrete		
						Average strength, psi	
	Angularity number	Roughness factor	Dynamic modulus (†10⁻⁶)	Specific gravity	Average cement factor	Flexural	Compressive
A	1	4.4	8.6	2.51	5.81	544	5944
B	5	5.3	8.6	2.52	5.83	623	6394
C	6	2.5	8.0	2.52	5.83	558	6287
D	8	2.2	8.0	2.55	5.85	611	6421
E	9	2.2	8.0	2.58	5.89	609	5978
F	2	9.4	10.7	2.61	5.92	609	6451
G	6	7.5	10.7	2.62	5.90	668	6513
H	9	13.1	10.9	2.64	5.95	688	6779
J	10	12.2	10.9	2.64	5.95	714	6752
K	9	16.6	3.3	2.62	5.93	584	6353
L	8	8.8	10.2	2.61	5.92	662	6951
M	10	11.6	10.5	2.71	6.02	674	6666
N	7	10.0	11.3	2.66	5.97	710 (8.4) †	6427 (4.3) †

*From author's Tables 3, 4, and 5. Concrete data are averages of all mixes and ages.
†Coefficients of variation, percent.

tionship is accidental, even though E may not have been a cause of the strength variations. Excluding K, the correlation coefficient is better, being 0.810, significant at a level of less than 1 percent.

But why should modulus of elasticity have such a well defined effect? It is probably related to the strength of aggregate particles, but the author points out, and we agree, that all of the aggregates were strong enough in relation to the mortar that their strength was probably of no significance.

A study of the data makes it immediately apparent that a good relationship exists between dynamic modulus and specific gravity of the aggregate. Perhaps that is a fortuitous agreement rather than a real relationship, but it exists. It is self-evident that specific gravity has a well defined effect on the characteristics of concrete through its influence on yield. The dynamic modulus versus specific gravity relationship is shown in Fig. G. With Aggregate K again omitted, the correlation coefficient is 0.826, which is significant at less than 1 percent.

That specific gravity, when allowed to influence the cement content of the concrete, should correlate with strength seems completely reasonable. That an excellent correlation did exist is shown by Fig. H. The correlation coefficient is 0.831 which, again, is significant at less than 1 percent. It should be emphasized that this relationship is a reflection of other factors — principally cement content but also fine-to-coarse aggregate ratio and consistency. In any event, whether by coincidence or otherwise, flexural strength was equally well related to both dynamic modulus and specific gravity of aggregate and these properties in turn were closely related to each other. It seems doubtful that either actually has a causal relationship, *per se*, to strength.

Thus, while the author has opened some highly interesting avenues for future investigation, we feel that he may have considerably over-emphasized the significance of the aggregate properties which he measured. That the high degree of simple correlation between flexural strength

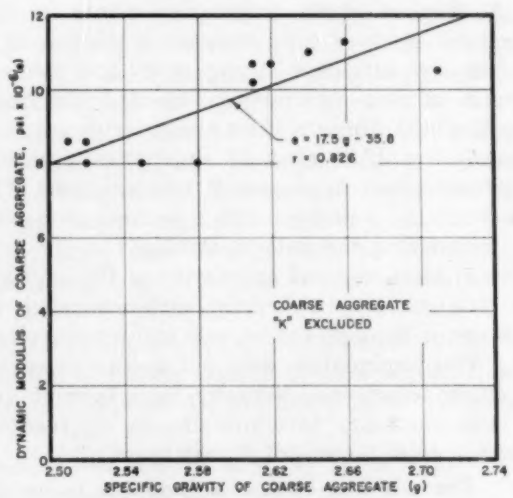

Fig. G — Relationship between modulus of elasticity and specific gravity of coarse aggregate

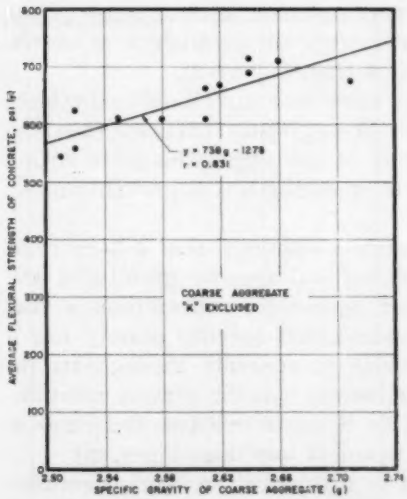

Fig. H—Relationship of concrete flexural strength to specific gravity of coarse aggregate

and aggregate modulus of elasticity may have been a reflection of other factors, not considered in the analysis, is suggested by our calculations. Strength was no better correlated with aggregate modulus of elasticity than with aggregate specific gravity. The latter two properties were found also to be closely correlated with each other, and it is easy to rationalize why specific gravity of aggregate, under the conditions of these tests, should have affected concrete strength. We are reluctant, therefore, to accept the conclusion that modulus of elasticity of aggregate affected strength even though it correlated well.

Aggregate surface texture would be expected to affect concrete strength, particularly in flexure. With all aggregates included, however, the correlation is just fair, producing a coefficient of only 0.488, significant at the 10 percent probability level. Here again, eliminating the eccentric Aggregate K (this time with an unusually high value for roughness factor) improves the correlation coefficient to an impressive 0.798, which is significant at less than 1 percent.

As implied above, angularity would be expected to correlate with strength in these tests because of the use of fixed proportions and the consequent influence of angularity and void content on workability and degree of over- or under-sandedness. The simple correlation coefficient for flexural strength versus angularity number was found to be a moderately low 0.551 when all aggregates were included and a slightly better 0.651 when Aggregate K was excluded. These values are significant at about the 5 percent and 2 percent probability levels, respectively.

Considering the data in the light of our approach, we feel that modulus of elasticity and angularity of the aggregate probably were related to strength only indirectly, either through mutual relations to other factors or through extraneous influences related to the methods of testing. This explanation does not appear to apply to the effects of surface texture, which may actually have been more than apparent, although it was necessary to eliminate one aggregate from the calculations to show a high degree of correlation.

The author's closure will appear in an early 1960 JOURNAL.

Discussion of a paper by Georg Wästlund

Use of High-Strength Steel in Reinforced Concrete*

By K. HAJNAL-KÓNYI, R. TAYLOR, A. ZASLAVSKY, and AUTHOR

By K. HAJNAL-KÓNYI†

In his very interesting paper Professor Wästlund has classified the requirements which a reinforced concrete structure has to fulfill under the following three main headings: (1) adequate safety against failure; (2) limited crack formation; and (3) limited deflection, i.e., sufficient rigidity.

Each of these conditions establishes a "ceiling" for the permissible steel stress and it is of course the lowest of the three ceilings which is relevant.

In the case of pure bending or of bending combined with a relatively small axial compression, adequate safety against failure can be obtained if the mode of failure is in tension, i.e., if premature failure in any of the other possible modes of failure enumerated in Professor Wästlund's paper is avoided. Under this assumption it can be shown that if the value of the so called "tension reinforcement index" $q = A_s f_y / b d f_c'$ does not exceed a certain limit (which depends on f_y, on the ultimate strain of the concrete in compression, and on the bond between reinforcement and concrete) there is practically no ceiling to the steel stress. If the two other conditions could be disregarded, even high tensile wire with an ultimate strength of 120 tons per sq in. could be used as ordinary (i.e., nontensioned) reinforcement at say half its strength. This, however, is not true because the two other conditions impose a much lower limit on the working stress. Reference is made to Fig. 6 in Reference 11 (Fig. A in this discussion) which represents the load-deflection lines of six beams of identical dimensions containing different percentages of various types and qualities of steel and demonstrates the significance of the three ceilings.

The ultimate load on beam 20H reinforced with 0.178 percent of high tensile wire which was fractured at failure was 32 percent higher than the ultimate load on the comparable beam 4H reinforced with 0.98 per-

*ACI JOURNAL, V. 30, No. 12, June 1959 (*Proceedings* V. 55), p. 1237. Disc. 55-75 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 6, Part 2, Dec. 1959 (*Proceedings* V. 55).

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cent mild steel although the cube strength of the latter was 20 percent higher. Yet beam 4H fulfilled both Conditions (2) and (3) until the maximum load was reached whereas beam 20H showed both excessive cracking and excessive deflection already at approximately 1/3 of the ultimate load. Thus for beam 4H the ceiling is the steel stress whereas for beam 20H the ceilings are determined by the crack widths and the deflection.

Professor Wästlund has dealt with the question of crack widths in detail. Briefly, it may be stated that because of the crack widths, steel of a yield point or proof stress of more than about 100,000 psi can hardly be used economically as nontensioned reinforcement, even if all possible precautions are taken regarding the distribution of the steel in the tensile zone.

Regarding deflection, one has to distinguish between instantaneous deflection caused by the application of the load and deflection under

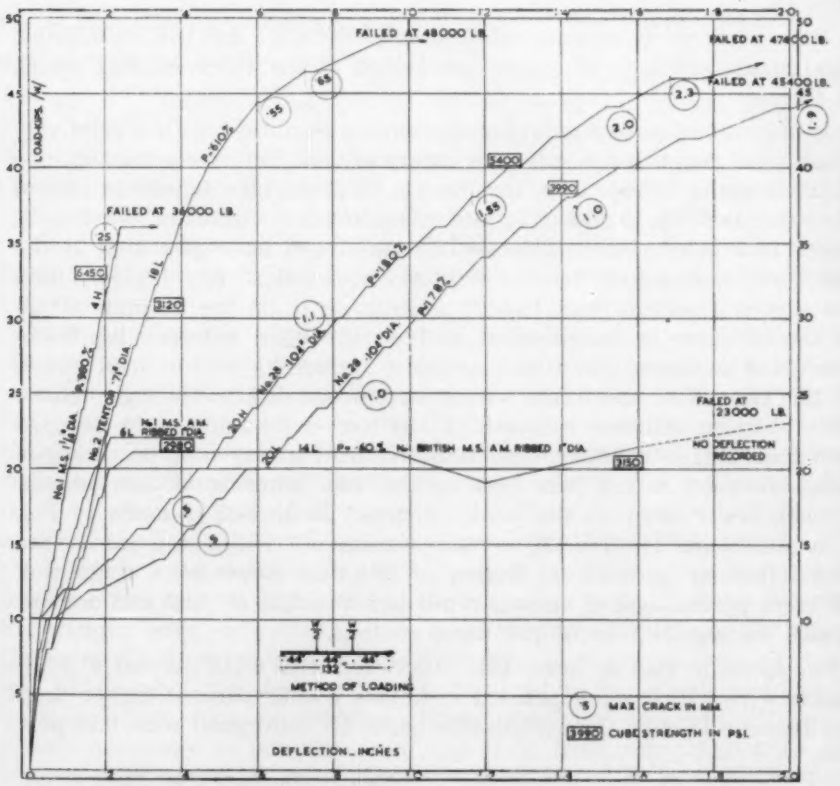

Fig. A—Typical load-deflection diagrams (Fig. 6, Reference 11)

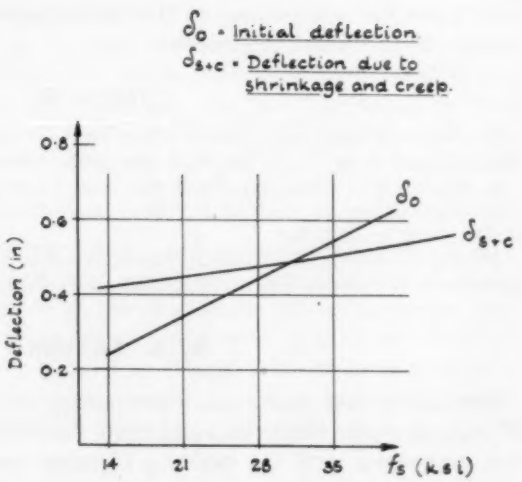

Fig. B — Relation between steel stress and deflection in a numerical example according to Professor Leonhardt

sustained loading. Instantaneous deflection caused by the dead load can be compensated by an initial camber. This also applies to a certain extent to the deflection caused by shrinkage and creep under dead load but not to deflection caused by live load.

In a paper read at the convention of the Deutscher Betonverein in Munich in May, 1959,[12] Professor Leonhardt showed on the basis of theoretical considerations that while the instantaneous deflection increased in the same proportion as the working stress in the reinforcement, the deflection under sustained loading was much less affected by the steel stress. This is illustrated in Fig. B which is reproduced from Professor Leonhardt's paper with his permission. (The metric system has been converted into lb and in.).

The writer has carried out comparative tests on 12 beams under sustained loading.[13] Six of these were reinforced with mild steel and kept loaded at a stress of 20,000 psi; six were reinforced with Tentor steel and kept loaded at a stress of 34,000 psi. While the average ratio of the instantaneous deflections of the two groups was 1.40, the average ratio of the total deflections after more than 2 years of sustained loading was only 1.15 which is a confirmation of Professor Leonhardt's theory.

Experience has proved that it is important to consider the deflection after a long period of loading even with mild steel at a working stress of 20,000 psi. The importance of the deflections increases with increasing working stress but if an adequate camber is provided to compensate for the influence of the dead weight the effect of an increased working stress as against that of mild steel is much less serious than would ap-

pear from the comparison of the instantaneous deflections. Further research in this field is desirable.

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13. Hajnal-Kónyi, K., "Special Reinforcements for Reinforced Concrete," *Rapport Final du Symposium*, RILEM, Liége, July 1958, pp. 117-180.

By R. TAYLOR*

The author has posed some interesting problems concerning the use of high-strength steels in reinforced concrete. One of these problems, that concerned with the bending of these steels, has recently been the subject of an exploratory series of tests carried out at the Building Research Station, England.

The problem arises with the possible use of high-strength steels for shear reinforcement in the form of stirrups. In such cases it would often be necessary to bend the bars to a smaller diameter than is required by standard specifications. In fact, in designing to the British Code of Practice BS CP.114, it is permitted for the provision of anchorage, to bend the stirrup around a bar of the same diameter (see Reference 14 for further discussion of this).

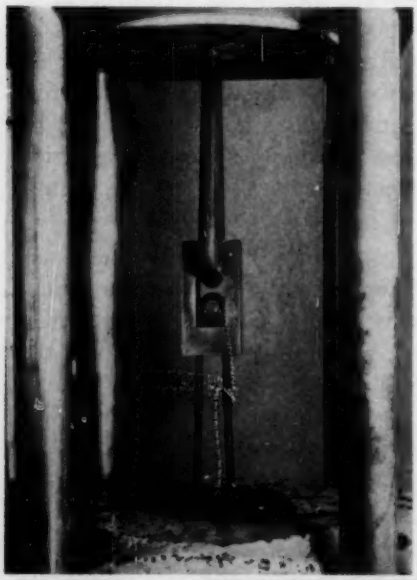

Fig. C—Testing arrangement

To obtain some data on the effect of bending high-strength steels around small diameters, comparative tests were made on four types of reinforcing bar produced in England: (1) a plain round ordinary mild steel bar (MS); (2) a plain round high-tensile steel bar, its high strength being due to its composition (HT); (3) a cold-worked round steel bar having the herringbone pattern of deformations, the bar having been cold worked by twisting and stretching in a similar way to the Danish bar de-

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scribed in Professor Wästlund's paper (T); and (4) a cold-worked steel bar produced by twisting a square mild steel bar (ST). Two sizes of each type of bar were tested: the round bars were ¼ in. and ⅜ in. diameter; the others were ¼ in. and ⅜ in. square. Specimens of the ¼-in. bars were bent using a normal type bending machine around pins of ⅜ in. and ½ in. diameter; specimens of the ⅜-in. bars were bent around pins of ½ in. and ⅝ in. diameter. Two specimens of each type were bent at each diameter.

It was decided to bend the bars at a lower temperature than normal laboratory temperature since it was considered that this would show up any differences in the effect of bending much more markedly. Accordingly the bars were stored overnight in a room at a temperature of 25 F. These, suitably wrapped, were then transported in small numbers to the bending shop. The actual temperature of the bars at the moment of bending was not obtained but generally this would not be much higher than 25 F.

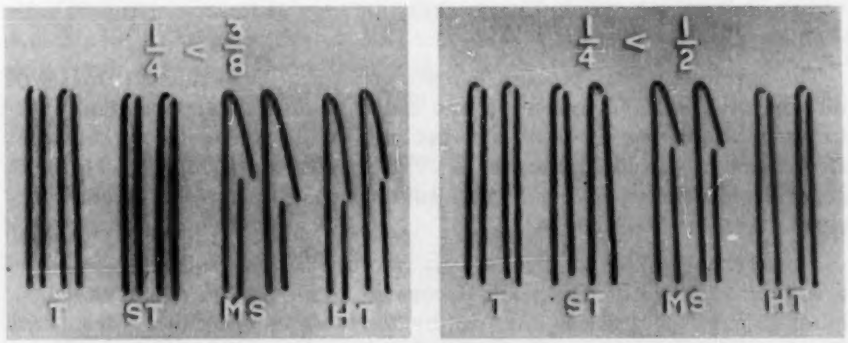

Fig. D(a)—¼-in. specimens after test. Left—Bars bent around a ⅜-in. pin. Right —Bars bent around a ½-in. pin

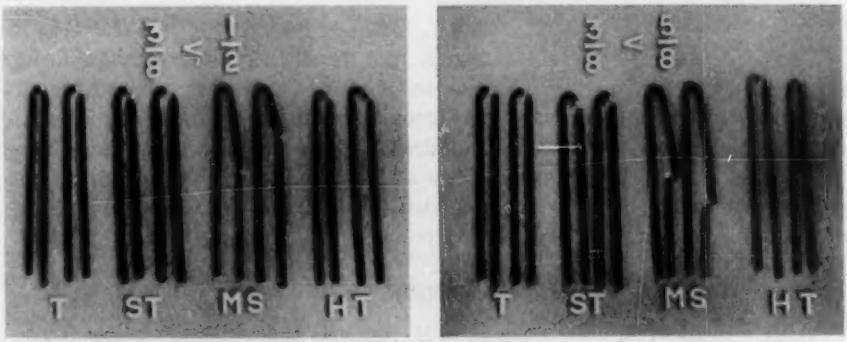

Fig. D(b)—⅜-in. specimens after test. Left—Bars bent around a ½-in. pin. Right —Bars bent around a ⅝-in. pin

TABLE A—RESULTS OF TESTS

Type of steel	Ultimate strength, psi		Load taken by specimen (percent of ultimate strength of straight bars)			
			¼-in. bar bent around:		⅜-in. bar bent around:	
	¼-in. bar	⅜-in bar	⅜-in. pin	½-in. pin	½-in. pin	⅝-in. pin
Mild (MS)	66,000	66,000	100	100	100	100
High tensile (HS)	106,000	108,000	98	96	96	98
Cold worked (T)	82,000	80,000	89	100	89	95
Cold worked (ST)	81,000	73,000	89	97	94	98

The bars were bent through 180 deg around the pin to form U-shaped test specimens. It must be noted here that (a) a perfectly symmetrical bend was not obtained in all cases, and (b) in none of the specimens was there any obvious sign of cracking of the steel at the bend. The strength of a specimen was determined by applying a load to the arms, the specimen being supported at the bend through a bar of the same diameter as that of the bend. The general arrangement for testing can be seen in Fig. C.

The specimens after the test are shown in Fig. D(a) and D(b). It will be seen that, without exception, the ordinary mild steel specimens fractured in one of the arms of the specimen, while the cold-worked steel specimens fractured at the bends. The specimens of high-tensile steel generally fractured at the bends, but in two of them the fracture occurred within the arms.

The average values of the loads at fracture for the pairs of specimens are given in Table A. These are expressed as the percentage of the combined strength of the two arms of the specimen as determined from test pieces cut from the same reinforcing bar. It will be seen that, with those specimens which fractured at the bends, there was generally some reduction in the carrying capacity, due to the bend, but that this reduction was never considerable.

Thus the tests appear to indicate that these high-strength steels do not suffer so great a loss of strength at bends that they should be excluded from use as stirrup reinforcement. However, no definite conclusions regarding this should be drawn from these tests which were only of an exploratory nature. It is also possible that variations in the ductility between batches of these steels might give considerably different results from those reported here.

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14. Hajnal-Kónyi K., Discussion of E. M. Rensaa's paper "Shear, Diagonal Tension, and Anchorage in Beams," ACI JOURNAL, V. 30, No. 12, June 1959 (*Proceedings* V. 55), pp. 1435-1438.

By A. ZASLAVSKY*

The paper deals authoritatively with a subject of great practical and economical importance, its limitations and future development.

High-strength steels

It may be of some interest to mention a few more European types of high-strength reinforcement.

A special type of cold-notched reinforcement steel was developed by Avakov in Russia[15] and the writer has also carried out tests with steel produced by a similar method.[16] The plain mild steel bar undergoes cold notching in two perpendicular directions in a single rolling process (Fig. E). Bars of small diameters can be fed directly from reels and coiled again. Cold notching produces deformed bars with a raised yield (proof) stress of 52,000 psi and more (depending on the quality of the virgin material) with excellent bond properties. At the same time, a gain of about 10 percent in length is obtained. Strain hardening takes place also in the small intervals between notches but sufficient ductility is retained. Another advantage is that (similar to cold-twisted bars, but unlike cold-stretched bars) the yield point is equally raised in tension as in compression, so that the steel may also be used as compression reinforcement.

A special high-tensile steel developed in Austria is the twin bi-steel (Fig. F). The steel consists of two plain round parallel high-strength bars of small diameter (⅛ to ⅜ in.) joined together by welded mild steel steps at 3 to 4-in. intervals. Bond is assumed to be provided by these steps, which also control cracks. No hooks are required and additional anchorage is provided by extra length of steel. The permissible tensile stress may (under certain conditions) be as high as 64,000 psi and the reinforcement ratio as low as 0.1 percent.

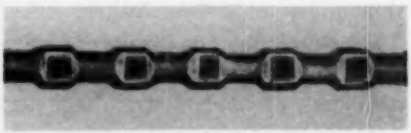

Fig. E—Cold-notched reinforcement

Fig. F—Austrian twin bi-steel

Another steel with a high permissible tensile stress (up to 57,000 psi) is the German Neptun steel[17] having a flat rectangular section (Fig. G). The steel is hot rolled, ribbed, twisted, and heat treated.

Fig. G—German Neptun steel

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TABLE B—COEFFICIENTS FOR CALCULATION OF CRACKS AND DEFLECTIONS BY THE RUSSIAN CODE (Data apply to rectangular beams with tensile reinforcement only)

$\alpha = 3\mu n$	ψ values for f_s, psi (Values in parentheses are kg per sk cm) 17,800 (1250)	21,400 (1500)	28,500 (2000)	35,600 (2500)	42,700 (3000)	K	j	C
0.05	—	—	—	—	0.39	22.8		
0.06	—	—	—	0.40	0.57	19.2		
0.07	—	—	—	0.49	0.68	16.0		
0.08	—	—	0.40	0.63	0.75	14.0		
0.10	—	0.40	0.58	0.76	0.86	11.8	0.86	0.63
0.125	—	0.47	0.73	0.85	0.91	9.6	0.85	0.60
0.15	0.44	0.63	0.82	0.90	0.94	8.0	0.84	0.57
0.20	0.65	0.78	0.90	0.94	0.97	6.0	0.82	0.52
0.30	0.82	0.89	0.94	0.97	0.98	4.3	0.79	0.46
0.40	0.88	0.92	0.96	0.98	0.99	3.4	0.77	0.42
0.50	0.90	0.93	0.96	0.98	0.99	2.8	0.75	0.38
0.80	0.93	0.95	0.97	0.98	0.99	2.2	0.71	0.30

Referring to the author's Fig. 3 for Tor steel, it seems rather coincidental that the nominal rupture stress should be equal to the yield (proof) stress.

A recent development is the *ribbed* Tor steel which does not require hooks.

Influence of concrete quality

While the influence on crack formation is clearly established for some of the factors, the influence of the quality of the concrete is only briefly mentioned in the paper and it is hoped the author may deal with this question in his closure.

However, on the basis of the author's paper and other sources[18] it appears that concrete quality has little influence upon the size of cracks, provided, of course, shrinkage is not present.

As regards the concrete, the writer feels that its plasticity (i.e., extensibility) rather than its strength f_r (see author's formula on p. 1245) is the decisive property. Preliminary tests performed by the writer seem to justify this contention. This does not contradict the statement about the strength of the concrete because its extensibility is little affected by its strength (as confirmed by Hognestad, Evans, Ros). Assuming a (hypothetical) concrete of 0.1 percent strain extensibility and ideal bond, the first cracks would appear only after a stress of about 30,000 psi has developed in the reinforcement — irrespective of the strength of the concrete. It may, therefore, be rewarding to produce a highly plastic concrete even at the expense of strength, the ultimate bending capacity of the beam being practically independent of the strength of the concrete. There arise, of course, additional questions of deflections and cracks caused by diagonal tension, but it may be expected that these would form in a similar manner.

Crack and deflection formulas

It may be noted that the 1955 Russian code also includes formulas (developed by Murashev) for the calculation of cracks and deflections.[19]

For rectangular beams the (mean) crack width is calculated from (author's notation used):

$$\omega = \psi\, a\, \frac{f_s}{E_s} \qquad (1)$$

Here the coefficient $\psi \leqq 1$ represents the tensile participation of the concrete in the spaces between the cracks.

The crack spacing a is calculated from:

$$a = 0.5\, Kn\, \frac{A_s}{\Sigma o} \qquad (2)$$

The 0.5 coefficient applies to deformed bars; for plain round bars the coefficient is 1.0.

$n = E_s/E_c$ = ratio of the elastic moduli
Σo = sum of the bar perimeters
In the case of round bars $A_s/\Sigma o = \frac{1}{4} D$ (D = bar diameter)

The coefficient K as well as ψ depend upon the value $\alpha = 3\mu n$ ($\mu = A_s/bd$ = reinforcement ratio) and the ψ coefficient depends in addition on the steel stress f_s.

Table B (abridged) shows the ψ and K values for rectangular beams with tensile reinforcement only. Places marked (—) indicate that cracks do not form. It can be seen that the product Kn in Eq. (2) is not influenced much by the E_c of the concrete.

Deflections are calculated with the aid of the (minimum) flexural rigidity B given by

$$B = \frac{C}{\psi} E_s A_s d^2 \qquad (3)$$

The coefficient C is included in the table. The rigidity B replaces the usual value EI of homogeneous sections. The effect of creep is accounted for by multiplying dead load with a factor $\Theta = 1.5 - 2.5$.

As a numerical example consider a simply supported rectangular beam of $l = 15$ ft span under a uniformly distributed load $w = 750$ lb per ft. The dimensions of the beam section are: $b = 4$ in.; $d = 14$ in.; and the reinforcement: two deformed round bars of ⅝-in. diameter ($A_s = 2 \times 0.31 = 0.62$ sq in.; $\mu = A_s/bd = 0.62/\ (4 \times 14) \approx 0.0115 = 1.15$ percent).

The elastic moduli are: $E_s = 30{,}000{,}000$ psi and $E_c = 3{,}000{,}000$ psi, i.e., $n = E_s/E_c = 10$. The $\alpha = 3\mu n = 3 \times 0.0115 \times 10 \approx 0.35$.

The maximum bending moment is:

$$M = \tfrac{1}{8}\, wl^2 = \tfrac{1}{8} \times 750 \times 14^2 \approx 18{,}400 \text{ in.-lb}$$

The steel stress can now be calculated from the usual formula:

$$f_s = \frac{M}{jdA_s} = \frac{18{,}400}{0.78 \times 14 \times 0.62} = 27{,}200 \text{ psi}$$

jd being the (reduced) arm of the resisting couple; $j = 0.78$ was found in the table for $\alpha = 0.35$. The other required coefficients, taken from the table, are: $\psi = 0.942$; $K = 3.85$; and $C = 0.44$. From Eq. (2) we now calculate the crack spacing:

$$a = 0.5\,Kn\,(A_s/\Sigma o) = 0.5 \times 3.85 \times 10\,(\tfrac{1}{4} \times \tfrac{1}{8}) = 3.0 \text{ in.}$$

and from Eq (1) the crack width:

$$\omega = \psi\, af_s/E_s = 0.942 \times 3.0 \times 27{,}200/30{,}000{,}000 \approx 0.0026 \text{ in.}$$

Calculating ω_{max} by the formula given by the author on p. 1247, we obtain approximately:

$$\omega_{max} = kD\left(\frac{I_c}{dA_s\,e_t}\frac{f_s}{E_s}\right)^{\frac{2}{3}} = 0.16 \times \tfrac{1}{8}\left(\frac{1100}{14 \times 0.62 \times 8.8} \times \frac{24{,}200}{30{,}000{,}000}\right)^{\frac{2}{3}} = 0.0051 \text{ in.}$$

Since ω_{max} may be larger than ω by 50-100 percent the agreement between the formulas is satisfactory.

The flexural rigidity of the beam equals, by Eq. (3):

$$B = C\,E_s\,A_s\,d^2/\psi = 0.44 \times 30{,}000{,}000 \times 0.62 \times 14^2/0.942 = 170 \times 10^7 \text{ lb-in.}^2$$

Thus the maximum deflection:

$$\delta = \frac{5}{384}\frac{wl^4}{B} = \frac{5(750/12) \times (15 \times 12)^4}{384 \times 170 \times 10^7} = 0.5 \text{ in.} = l/360$$

Assuming the dead load to be 400 lb per ft, the live load 350 lb per ft, and $\theta = 2$, the deflection will increase to:

$$\delta' = (2 \times 400 + 350)/750 = 0.5 \times 1.54 = 0.77 \text{ in.}$$

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AUTHOR'S CLOSURE

Mr. Taylor shows that high-strength steel with a high yield point can have good bending properties. A high strength can be attained together with a good bending property by suitable metallurgical process, but for this bending property to be reliable, it will require additional expenses.

Dr. Hajnal-Kónyi says that under certain assumptions it can be shown

that if the value of the so called "tension reinforcement index" does not exceed a certain limit, there is practically no ceiling to the steel stress. This underlines what the author says on p. 1242, with the same reservations, that if the yield of the reinforcement is the deciding factor for failure it is possible to increase the permissible stresses approximately in proportion to the yield point, i.e., under the reservations made without ceiling.

Dr. Hajnal-Kónyi also adds new facts and presents interesting aspects, particularly with reference to deflection.

Mr. Zaslavsky wants the influence of the quality of the concrete on the crack formation discussed in more detail. The author had the intention with his article only to mention the most important problems arising in the use of high-strength reinforcing bars. As mentioned in the article, several of these problems are serious in that there is still a lack of experimental research, e.g., regarding the splitting of the concrete along deformed bars or the splitting at bends. The influence of concrete quality on crack formation must however be considered a problem of secondary order. This is due to two reasons.

(1) The formula for the crack spacing a_m on p. 1245 includes the ratio between the concrete tensile strength and the bond strength between the concrete and the steel in question. It is known that the bond strength increases with the concrete tensile strength, even if not proportionally. In most theories or empirical formulas the ratio is assumed to be a constant, indicating that the concrete quality has only a small influence on the crack spacing.

(2) The crack width is dependent mainly on the crack spacing and the steel stress. The surrounding concrete will as a secondary effect restrain the opening of cracks in some proportion to its quality. The effect of shrinkage will on the other hand increase the opening of cracks, often in some proportion to the concrete strength, because a high strength concrete, if attained through higher cement content, will have a higher shrinkage value. These two effects thus are counteracting each other, which means that concrete quality will have a small in fluence also on the crack width.

Mr. Zaslavsky means that the extensibility of the concrete rather than its strength is a decisive property as regards crack formation. According to what is said the extensibility must have a small influence.

The review of the Russian code regarding calculation of cracks and deflection is interesting.

Discussion of a paper by R. Shalon and M. Raphael:

Influence of Sea Water on Corrosion of Reinforcement*

By EDWARD A. ABDUN-NUR, J. BROCARD, and AUTHORS

By EDWARD A. ABDUN-NUR†

The authors have made a worthwhile contribution that will be of great help in solving or in combatting the detrimental effects of sea water on the embedded reinforcement in the concrete. The use of sea water in concrete mixes is a problem which many construction jobs are having to cope with, due to the lack of fresh water in some of the far flung areas in the world, where construction is becoming an important factor in the economic developments being carried out.

In connection with this general subject, it might be pertinent to present some experiences and observations with which the writer is familiar. A great deal of the gravel and sand used in making concrete for buildings along the eastern coastal strip of the Mediterranean comes from the sea, and is therefore, on drying, coated with a thin layer of the salts found in the sea water in the case of gravel, and has a small amount of sea water left in the moist sand. It is generally used without any further washing. Fresh water is, however, used in the mixing in most instances. The writer has observed concrete made in this manner that has been in service for many years, and he has used it in important structures. An examination of these structures after they had been in service over 20 years disclosed no outer evidence of distress of any kind in the concrete itself, or of cracking due to the swelling of corroded reinforcement.

The writer would assume that the lack of distress observed under such circumstances is probably due to the eventual low concentration of the salts in the total mix. Yet, Table 5 indicates that the low salt concentrations may at times be more detrimental than higher concentrations. It would be of interest to know whether the authors know of similar cases and the explanation they may have for the satisfactory behavior of concrete under such circumstances.

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By J. BROCARD*

The study made by Mesdames Shalon and Raphael was interesting to me as we did not make a systematic study on the subject they treated. I would like to point out, however, that the French NF-P18-302 Norm limits the content of dissolved salts to 15 g per l in water to be used for mixing concrete for reinforced concrete structures.

It is really the lack of laboratory studies that led to the arbitrary choice of this figure corresponding to a concentration between pure water and the average saltiness of the seas that border France.

In this sense, the initiative of the authors is interesting because it is research of this kind that will enable the drawing up of more exact "Normes et de Cahiers des Charges" (Norms and Conditions).

I noted particularly the notion of pH inhibition and variations due to the concentration of dissolved salts in water. In this regard I believe that the results of Fig. 1 correspond to concentrations much greater than normal.

The study shows in a significant manner the influence of aeration on corrosion of reinforcement and, thereby, the importance of the concrete's denseness and the thickness of the covering over reinforcement — which are primary factors.†

AUTHORS' CLOSURE

The authors are gratified by the interest shown by Messrs. Abdun-Nur and Brocard and wish to thank them for their valuable comments.

In reply to Mr. Abdun-Nur's remarks it may be said that in this country no sea gravel is being used for concrete. Sea sand has, as a rule, been taken from the shore at a distance from the sea where the salts had been washed away by the rain. Lately, however, sand taken directly from the sea has come into use. The authors have also been perturbed by the possibility of the salt left in the sand promoting corrosion and have therefore carried out tests on a number of fine coarse sea sand specimens to determine the amount of salts retained.

In all specimens, the chlorine content proved to be low, varying between the limits of 0.004 to 0.06 percent Cl or 0.07 to 0.10 percent calculated as NaCl. This would correspond to some 0.03-0.4 percent sodium-chloride in the mixing water. Such quantities of salt hardly reduce the pH value and have practically no influence upon the corrosion of reinforcement.

In concrete made with both sea sand and sea gravel the total amount of salt is greater, though the gravel contains, pound for pound, less salt

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†Brocard, J., "Corrosion of Steels in Reinforced Concrete," *Annales*, Institut Technique du Batiment et des Travaux Publics (Paris), Concrete and Reinforced Concrete Series No. 47, June 1958 (in French).

than the sand, because of its relatively small surface. However, in dense concrete, in which carbonation may be assumed to reach only a few millimeters, the pH value of the layer enveloping the reinforcement, though somewhat reduced by the salt (see Fig. 1), should remain high enough for inhibition of corrosion. This was probably the case in the structures mentioned by Mr. Abdun-Nur.

It should, however, also be pointed out that the fact that "no outer evidence of distress was disclosed" does not necessarily imply that no corrosion of reinforcement has taken place. No such evidence would become visible if the rust was of the "general" type, similar to that which developed in the 1:8 mortar specimens investigated by the authors (Fig. 3).

The authors are in full agreement with Mr. Brocard as regards the importance of the density of concrete. As to the depth of covering, however, its significance seems to be limited to cases where the density is insufficient. Tests show clearly that when the concrete or mortar around the reinforcing steel is sufficiently dense, even a thin cover is enough to protect the reinforcement.[9] It is, nevertheless, admitted that under ordinary conditions of work when good compaction of the concrete cannot be ensured, it is good practice to specify a sufficient depth of covering to reduce the danger of damage to the reinforcement.

Similarly, in the light of results obtained in the investigation conducted by the authors, it appears that the limitation of salt content in the mixing water to 1.5 percent, provided by the French standards NF-P18-302, is not sufficient. In air exposed concretes of imperfect density, the penetrating air would both reduce the pH and increase the supply of oxygen, producing conditions under which 1.5 percent of salt may well be within the most undesirable range of salt concentration (see Table 5).

The remark of Mr. Brocard concerning the results shown in Fig. 1 is not clear, as the effect of the normal range of salts concentration upon pH values is included in the curves presented.

Discussion of a paper by Bruno Thürlimann:

Fatigue and Static Strength of Stud Shear Connectors*

By I. M. VIEST and AUTHOR

By I. M. VIEST†

A review of the literature pertaining to composite concrete and steel beams shows that a large number of different types of shear connectors has been proposed and used over the past 40 years. However, until publication of the 1957 *Standard Specifications for Highway Bridges* of the American Association of State Highway Officials no generally accepted design procedure was available in this country for the design of shear connectors. The 1957 AASHO specifications corrected this situation through incorporation of design formulas for three most commonly used connectors: channels, spirals, and studs. The formulas were derived from experimental data evaluating the three types of connectors on a comparable basis.[9]

Professor Thürlimann chose to disregard this comparable approach and, on the basis of rather limited experimental evidence, proposed new formulas for the design of the L-shaped ½-in. stud connectors. The purpose of this discussion is to compare the static test data presented by Professor Thürlimann with the results of other tests and to raise a question concerning the interpretation of the results of the fatigue tests.

Static tests

Prior to the tests reported by the author, static tests of stud shear connectors were made with 22 push-out specimens[2,10] and with 26 plate-reinforced concrete beams.[11] The push-out specimens were similar to those employed by the author, the major difference being the prevention of the formation of bond between the slabs and the steel beams, casting of slabs in horizontal position, location of all connectors at one cross section of the steel beam, and virtual absence of slab reinforcement. Furthermore, all of the earlier tested studs were straight with an enlarged head, and their welding ends were fluxed with granular flux. The plate-reinforced concrete beams were made of a hot rolled steel plate ¼ in. thick, connected by stud connectors to 6 or 9 in. of concrete cast on top of the plate. Except in the negative moment regions of re-

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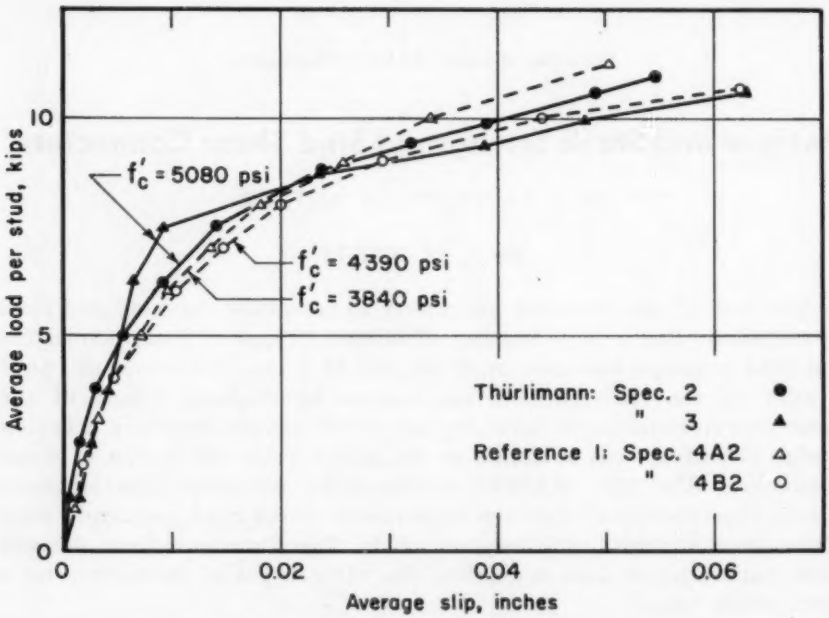

Fig. A—Load-slip curves for ½-in. diameter studs

strained beams, the concrete was unreinforced. No provision was made to prevent the formation of the natural bond between the concrete and the steel plate.

Two of the push-out specimens made with ½-in. diameter studs and four made with ¾-in. diameter studs may be compared directly with author's specimens tested with static loads. The dimensions of the studs and the properties of the materials are listed in Table A for all comparable specimens. Furthermore, the data on the yield strengths given in Reference 1 and the typical stress-strain relationship in Reference 2 indicate that the stud steel used in the two investigations had similar mechanical characteristics.

The load-slip characteristics of the specimens with ½-in. diameter studs are compared in Fig. A and the residual slip data are presented in Fig. B. The only difference between the author's and the earlier tests can be noted in the central portion of the load-slip curve of Specimen 3. The reason for this difference is unknown; however, the shape of the full load-slip curve for Specimen 3 presented in Fig. 12 suggests the possibility of accidental overloading prior to the conduct of the test.*

*In the writer's opinion, the comparison with the load-slip curve for the bridge test (Fig. 12) is not valid since the slips in the bridge test were retarded by the heavier connection on the adjacent beam (see Fig. 8 in Reference 1).

TABLE A—PROPERTIES OF PUSH-OUT SPECIMENS

Specimen No.	Stud dimensions		Strength of stud steel f_s', psi	Concrete strength f_c', psi
	Diameter d, in.	Length H, in.		
2	0.50	3.50	70,250*	5080
3	0.50	3.50	70,250*	5080
4A2	0.50	4.00	70,700	3840
4B2	0.50	4.00	70,800	4390
1	0.75	4.00	70,850*	5080
6A2	0.75	3.90	69,900	3870
6B2	0.75	3.90	70,400	4240
6A4	0.75	4.00	67,700	3360
6B4	0.75	4.08	69,300	3260

*Average values from Reference 1.

Parallel comparisons for specimens with ¾-in. diameter studs are presented in Fig. C and D. It will be noted that, although the shapes of the curves for all specimens are similar, the data for Specimen 1 fall consistently below those for the other specimens in spite of the approximately 20 percent higher concrete strength. This difference is particularly marked for residual strain curves. Although again no explanation of the differences is available in the test data, a number of causes can be suggested, such as collection of water under the studs due to vertical casting position, imperfect compaction of concrete around the studs, or imperfect welds, all of which may be considered an experimental error.

In the earlier tests of push-out specimens, the useful capacity of stud connectors was evaluated on the basis of the residual slip curves. For specimens with marked change in the rate of residual slip increase, the useful capacity was taken as the load corresponding to this change; for specimens with gradual change in the rate of residual slip increase, the useful capacity was taken as the load corresponding to the residual slip of 0.003 in. In the tests of plate-reinforced beams, the useful capacity of stud connectors was selected as that at which there was an abrupt

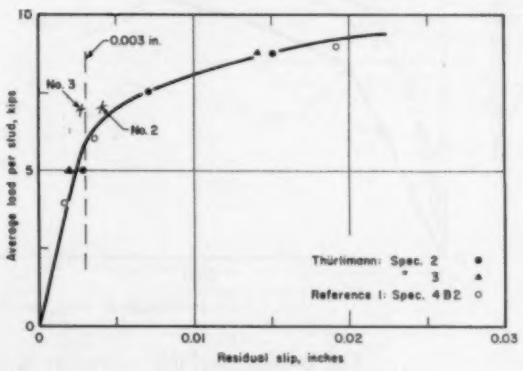

Fig. B—Residual slip curves for ½-in. diameter studs

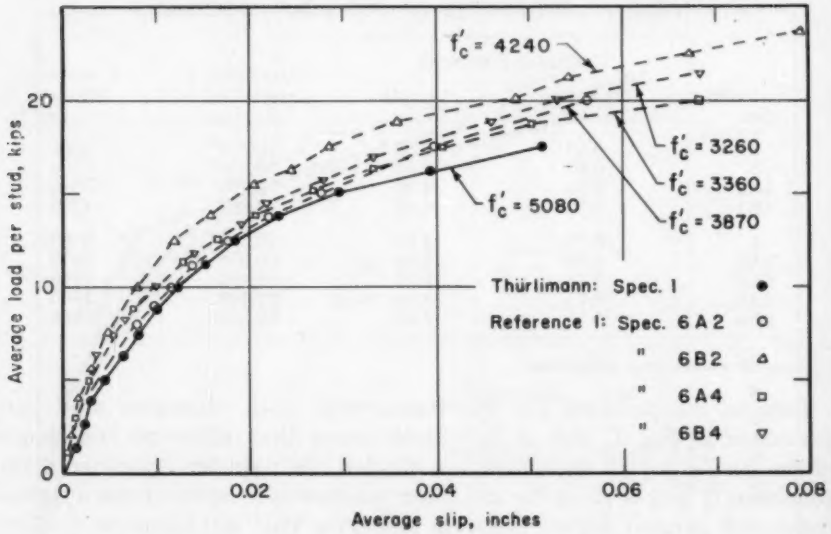

Fig. C—Load-slip curves for ¾-in. diameter studs

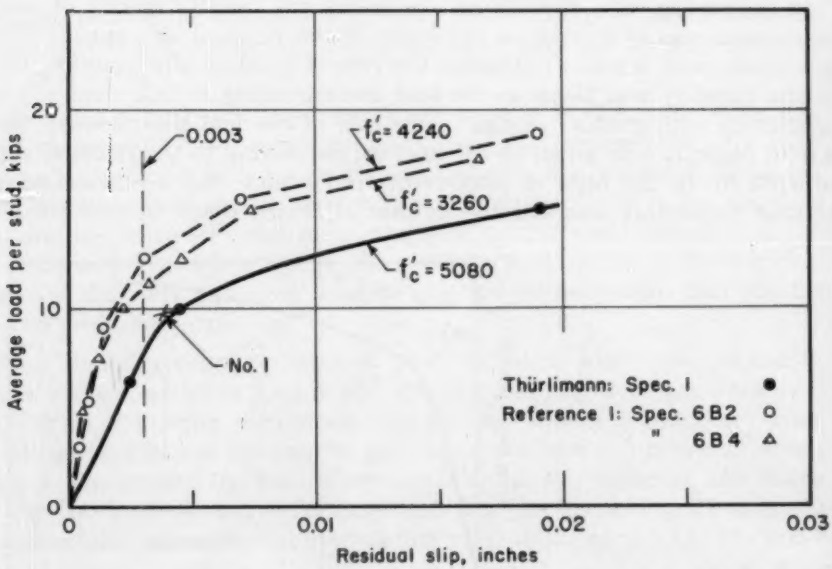

Fig. D—Residual slip curves for ¾-in. diameter studs

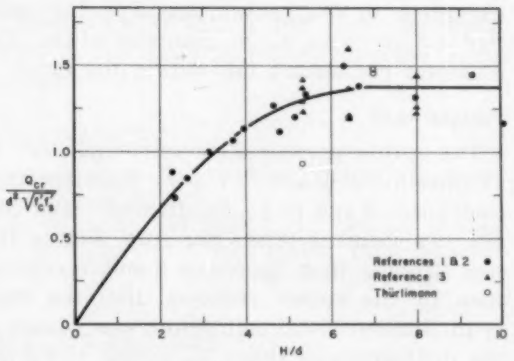

Fig. E—Critical capacity of stud shear connectors

change in the rate of slip. The loads considered as the useful capacity of the stud connectors and designated as critical loads are given in References 2, 10, and 11 for 15 push-out tests and 11 plate-reinforced beam tests. The data cover stud diameters ranging from 0.5 to 1.25 in.

The critical load for author's Specimens 1, 2, and 3 determined in the same manner as for the earlier push-out tests are indicated in Fig. B and D by cross marks. It will be noted that all three determinations are based on the change in the rate of residual stress increase; had the loads been determined on the basis of 0.003 in. residual slip, the values for all three specimens would have been somewhat lower. The critical loads are as follows:

Specimen 1: $Q_{cr} = 9.8$ kips
Specimen 2: $Q_{cr} = 7.0$ kips
Specimen 3: $Q_{cr} = 6.9$ kips

All 39 critical loads are presented in the dimensionless plot in Fig. E. It will be noted that the test data form a well defined band. The only

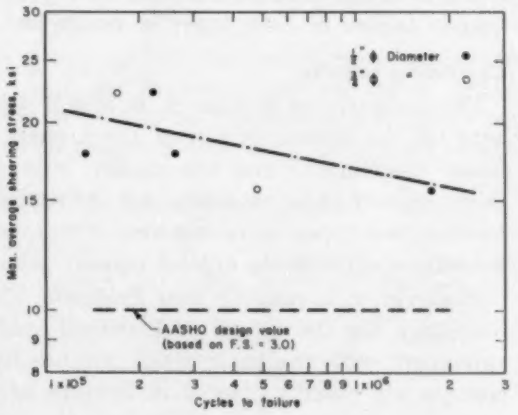

Fig. F—Fatigue tests of push-out specimens

exception to the general trend is the author's Specimen 3 (circle at $H/d = 5.33$) with ¾-in. diameter studs. The data for L-shaped ½-in. diameter connectors fall within the band.

Fatigue tests

The author presented direct comparisons between the fatigue strength of Specimens 4 and 7 (½-in. diameter stud connectors) with that of Specimens 9 and 10 (¾-in. diameter stud connectors) on the basis of the average shearing stress per stud. Noting that on this basis Specimen 4 was stronger than Specimen 9 and Specimen 7 was stronger than Specimen 10, the author reasoned that the higher fatigue strength of the ½-in. diameter stud connectors was caused by the smaller diameter and the differences in shape (L versus straight). To check on this tacit assumption of the differences in the fatigue strength, the writer prepared the plot shown in Fig. F including data for all author's specimens except for Specimen 8 tested with incremental loading. Judging on the basis of Fig. F, it appears to this writer that the differences quoted by the author may have been just as easily the result of the natural scatter in the test data as the result of the differences in the characteristics of the two types of connectors.

AASHO specifications

The AASHO specifications include two formulas for the design of stud connectors. These formulas represent a straight-line approximation of the curve shown in Fig. E. To obtain the working load for a stud connector, the formulas are divided by a factor of safety (FS), the value of which varies ordinarily between 3.0 and 4.0. Thus the maximum working load may be computed approximately with $FS = 3.0$; the corresponding average shearing stress for the specimens tested is shown in Fig. F. It can be seen that in the particular cases considered, the AASHO design procedure would have provided an ample safety against fatigue failure of both types of connectors.

Concluding remarks

The comparisons in Figs. A, B, and E show no substantial differences between the characteristics of the L-shaped connectors tested by Professor Thürlimann and the earlier tests of straight connectors. Thus there appears to be no reason for the use of different design procedures for the two types of connectors. The procedure given in the current AASHO specifications applies equally well to both types.

However, it is possible that Professor Thürlimann proposed the new procedure for the design of L-shaped stud connectors because of disagreement with the basic design philosophy on which the current procedures are based rather than because of some differences in the be-

havior of the two types of connectors. If this is the case, the writer would appreciate an elucidation of the basic principles involved in the formulas proposed by the author.

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AUTHOR'S CLOSURE

Dr. Viest has compared the results of the static push-out tests with results of similar tests made by him. The author would like to add three additional tests conducted independently by Mr. I. A. Benjamin of the Granco Steel Products Co., St. Louis. Table B, supplementing Table A, lists the properties of the specimens. Each contained only two straight studs of 3/4 in. diameter, 4 in. high with upset head, one on each side of the beam. They were welded manually to the beam, but care was taken to obtain a weld substantially equal to one produced by the stud welding process. Specimen 1B failed by shearing of one stud, the other two specimens by diagonal tensile cracks in the concrete and yielding of the studs. The results are shown in Fig. G and H, complementing Fig. C and D. It appears that neither Mr. Benjamin nor the author were able to reproduce the results obtained by Dr. Viest. In consideration of these additional results, Dr. Viest's suggested explanations concerning the results obtained on Specimen 1 with the 3/4 in. diameter straight studs are not substantiated. The author would like to point out once more that the speed of testing has considerable influence on the slip. In his static tests, readings were taken only after a waiting period to let load and slip stabilize.

Fig. I adds to Fig. E the results from Mr. Benjamin's tests. Again the author's test on Specimen 1 averages the other tests on 3/4 in. studs. In connection with this figure it should be mentioned that Dr. Viest has arbitrarily selected H as the length of the ½ in. diameter L-studs.

TABLE B—PROPERTIES OF PUSH-OUT SPECIMENS

Specimen No.	Stud dimensions		Strength of stud steel f_s', psi	Concrete strength f_c', psi
	Diameter d, in.	Length H, in.		
8A	0.75	4.00	100,000	4200
1B	0.75	4.00	100,000	3650
2B	0.75	4.00	100,000	3650

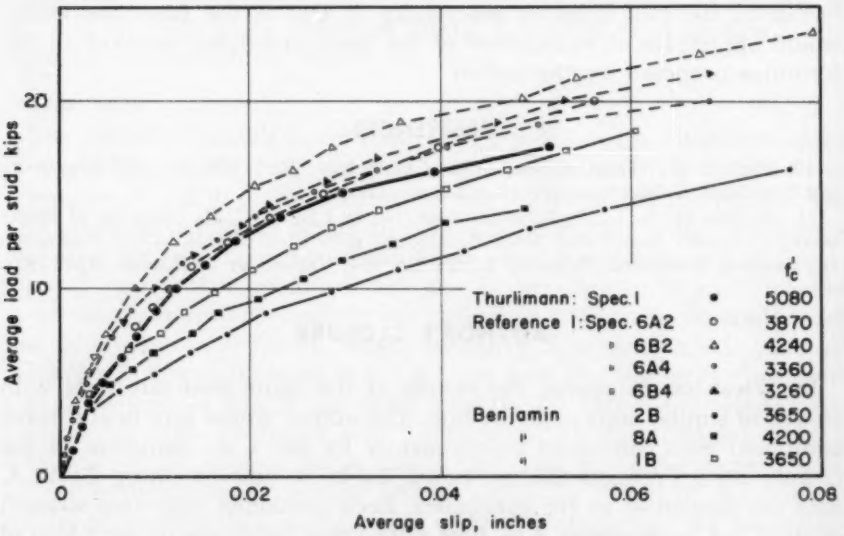

Fig. G—Load-slip curves for ¾ in. diameter studs

Ordinarily he uses H as the height of studs. If the actual height of the L-studs, equal to 2.25 in. is used, the open circles at $H/d = 7$ in Fig. I and E, respectively, would plot at $H/d = 4.5$.

In summary, there exists no major discrepancy as far as the ½ in. diameter studs are concerned. Nevertheless, it should be pointed out that the ½ in. L-studs of 2.25 in. height showed a better performance than the ½ in. straight studs, 4 in. high, with upset head, tested by Dr. Viest. A disturbing discrepancy exists for the ¾ in. straight studs with upset heads.

With regard to the fatigue tests, Dr. Viest disposes of the differences between the performance of the ½ in. L-studs and ¾ in. straight studs by explaining the discrepancy as natural scatter. The author agrees that the results show considerable scatter which is, however, by no means unusual in fatigue testing of structural components. Nevertheless, the fact stands that for the two stress levels at which both types were tested the ½ in. L-studs showed a longer fatigue life as may be seen from Fig. F. In addition, this figure shows clearly that the AASHO specifications which are based on Dr. Viest's studies lead to extremely conservative values as far as fatigue is concerned. Furthermore, Dr. Viest has arbitrarily taken a factor of safety (FS) = 3. The AASHO specifications recommend the value 4 in lieu of computing the factor by a given formula. For most designs the AASHO-line would, therefore, be rather at an average stress of 7.5 ksi, leading to an even greater unnecessary safety against fatigue.

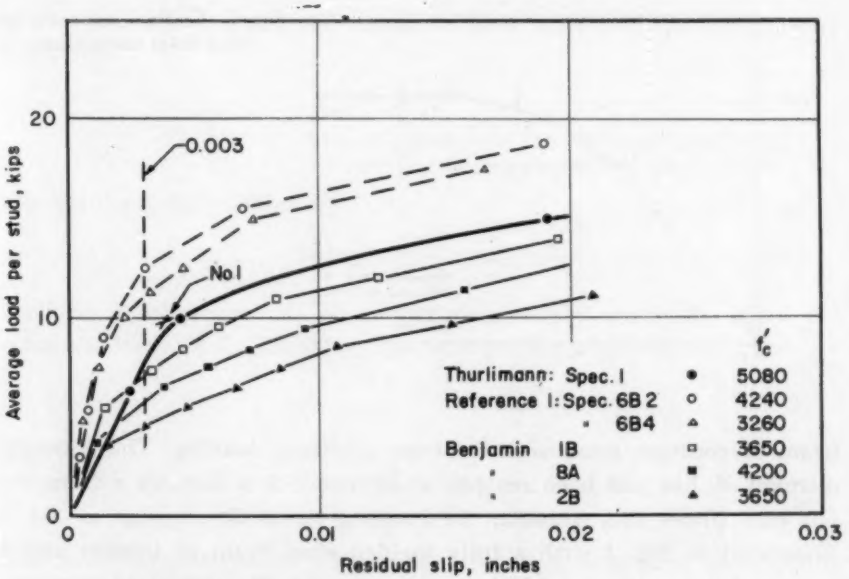

Fig. H—Residual slip curves for ¾ in. diameter studs

Finally the author would like to answer the discusser's question concerning a possible disagreement with regard to the discusser's basic design philosophy of composite beams. It may be well to recapitulate the four assumptions made by Dr. Viest:

1. He proposes "to select a factor of safety for the design of shear connectors capable of guaranteeing composite action up to the full flexural capacity" (Reference 9, p. 10).

2. "The useful load capacity Q_{uc} must be equal to the load acting on a shear connector at ultimate load." (Reference 9, p. 10).

3. "The useful load capacity is the load at which either the steel of the connectors begins to yield or the inelastic deformations of the concrete begin to increase rapidly." (Reference 9, p. 8).

4. The force acting on a shear connector at ultimate load is given by the elastic formula (Reference 9, Eq. (6b):

$$Q_{uc} = \frac{V_u m}{I} p \qquad (1)$$

V_u = transverse shear at ultimate load
m = statical moment of the transformed concrete section
I = moment of inertia of the composite section
p = spacing of connectors

The author is in full agreement with Point 1. However, the other three points are by no means as settled as Dr. Viest may think. Firstly, Eq. (1) is an *elastically* derived formula which he applies without hesitation to an *ultimate* load condition. It will be shown that such a mixture does not even lead to equilibrium. Fig. J shows a simply supported composite

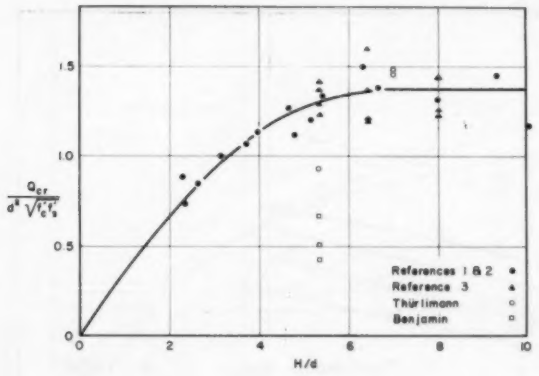

Fig. I—Critical capacity of stud shear connectors

beam of constant cross section under arbitrary loading. The ultimate moment M_u has just been reached at Section *S-S*, a distance a from the left end. Under this situation the bending stress distribution at *S-S* is illustrated in Fig. J with a fully yielded steel beam in tension and a rectangular compressive block in the concrete. The resulting compressive force in the concrete is

$$C = M_u/d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where d is the distance between the compression and tension resultants. Considering now the slab to the left of *S-S* as a free body, it must be in equilibrium under this compressive force C and the shear transmitted

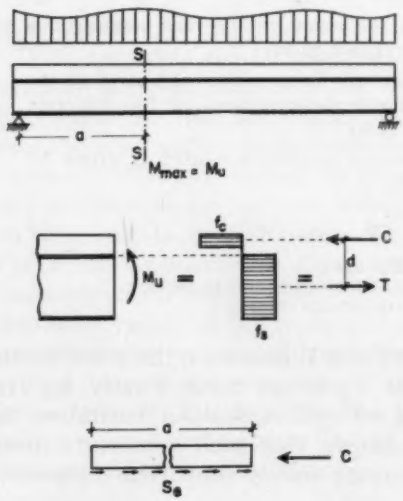

Fig. J — Horizontal shear at ultimate load

by the connectors. Using the elastic formula on which Eq. (1) is based it follows

$$S_e = \frac{V_u m}{I} \quad \text{(3)}$$

with S_e = shear flow per unit length at ultimate load, using elastic formula

The resulting force opposing C is

$$\int_0^a S_e dx = \frac{m}{I} \int_0^a V_u dx = \frac{m M_u}{I} \quad \text{(4)}$$

To have equilibrium, the above resultant should obviously equal the compressive force C. However, this is not the case such that

$$\int_0^a S_e dx - C = \frac{m M_u}{I} - \frac{M_u}{d} < 0 \quad \text{(5)}$$

The ratio

$$\int_0^a \frac{S_e dx}{C} = \frac{md}{I} \quad \text{(6)}$$

is a direct measure of the unbalance of forces. It is always considerably below unity. Table C lists a few specific values showing that the application of the *elastic* Eq. (1) used by Dr. Viest to calculate the shear connector forces at *ultimate* load underestimates the total horizontal shear by as much as 33 percent. Furthermore, the ratio md/I is not constant such that the situation can not be saved by the application of a correction factor. The least that must be required from a design procedure is that it leads to equilibrium. Obviously Dr. Viest's procedure does not fulfill this fundamental requirement.

Concerning Assumptions 2 and 3 it should be realized that Dr. Viest proposes to maintain the shear connection in essentially an *elastic state* at a point where bending has fully *plastified* part of the steel beam and the concrete of the slab is at the instant of *crushing*. To the author's knowledge, Dr. Viest has never presented any experimental nor theoretical evidence for these assumptions. The single beam test, B21W, of Reference 12 with a weak shear connection can certainly not be taken as sufficient substantiation. It is the author's opinion that the arbitrary

TABLE C—EXAMPLES OF md/I RATIOS

Specimen	B24W*	B24S*	B21S*	B21W*	Bridge†	Lehigh‡ tests
md/I	0.72	0.71	0.73	0.73	0.72	0.67

* From Reference 12.
† From Reference 1.
‡ Recent tests at Fritz Engineering Laboratory, Lehigh University.

selection of the useful load capacity of shear connectors together with the assumption that it should not be exceeded at ultimate load leads to a substantial overdesign of the shear connection. The performance of the beam is not improved at all as an overdesigned shear connection will not result in a higher bending strength.

Recently the Joint ACI-ASCE Committee on Composite Construction of which Dr. Viest is chairman, reviewed the situation concerning the design of shear connectors. The following passage is quoted from the paper "Development of Tentative Design Recommendations for Design of Composite Beams and Girders in Buildings," presented by Mr. P. Page at the ASCE Convention, 1959, in Washington, D. C.:

> The committee chose the useful capacities given in the AASHO specifications as the basis for determining the recommended allowable load formulas for various connectors. However, in using the values of useful loads the committee realizes their conservative nature particularly in relation to design for static loading, and is therefore considering the use of a 10 to 20 percent lower factor of safety than 2.4. More research is needed and the committee intends reviewing this factor of safety with an eye towards further downward revision if indicated by tests.

The author feels satisfied that a revision of the entire design of shear connectors is considered. Such a study together with appropriate tests is presently in progress.

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